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Page 513, in the footnote, change "Ward B. Maurer, Assoc. M. Am. Soc. C. E." to "E. R. Maurer"; and change "page 36" to "page 37."

Page 1404, Table 11, Column R, line (2)*b*, change "+ 0.510" to "+ 0.535"; and line (3)*b*, insert "+ 0.510."

Page 1807, lines 7 and 8 of paragraph two should read "* * * Monongahela River; construction of the Duquesne Way Elevated Railroad along the Allegheny River in downtown Pittsburgh, including warehouses and delivery yard; East * * *."

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Standards of Professional Relations and Conduct

BY

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STANDARDS OF PROFESSIONAL RELATIONS AND CONDUCT

BY DANIEL W. MEAD,[1] PAST-PRESIDENT AND HON. M. AM. SOC. C. E.

SYNOPSIS

This paper is intended to cover, so far as practicable, the personal and ethical relations of the engineer in all ordinary positions and branches of the profession.

The paper has been written more particularly for the younger men of the profession and with the hope that it may supply them with some information, usually not available in college, as to what their action and conduct should be in their professional relations in practical life. The principles enunciated are those confirmed by the fifty-six years of experience of the writer in the engineering profession. All are principles that the writer believes should not only be acknowledged but actually put into practice in professional service. Their genuine acceptance and practical application would greatly improve the standing of the entire profession, would add to the value of its services, and would greatly strengthen its influence on public opinions. Their application will also accomplish the greatest measure of personal success and personal satisfaction.

The writer first discusses the vital relation of good principles and good conduct to success in life and attempts to analyze the various characteristics necessary for professional success. This is followed by a suggested code of courtesy and personal conduct. Then follow suggested codes for ten departments of engineering activity. The writer recognizes the fact that this paper does not cover, completely, the entire field of engineering. It is believed, however, that a careful consideration of these suggested codes will make obvious to the engineer what his course of conduct should be in almost any field not specifically covered. It is not suggested that any such involved code be adopted as the official code of the Society, but it is hoped that the discussion will be sufficiently broad and complete to determine whether or not the principles outlined meet the approval of the profession. From the paper and its discussion, it is hoped that an unofficial code may be developed sufficiently

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complete so that any engineer in any ordinary position or in any usual line of work may determine the opinion of the profession as to the proper course of action which should be pursued. A Bibliography of the Literature on Ethics and Human Engineering is added as an Appendix.

INTRODUCTION

While President of the Society in 1936 the writer pointed out that most engineering ethical codes seemed to apply almost exclusively to the engineer in general practice and not to the more than 90% of the profession who are public or private employees.[2] He suggested that it would be desirable to prepare an unofficial code of ethics which would apply directly to all members of the engineering profession in all its ordinary diverse branches and positions. During 1936 he discussed before many of the local sections and student chapters the question of professional relations and professional ethics and found the subject apparently of considerable interest.

The subject was referred by the Board of Direction to its Committee on Professional Conduct who reported that they believed it undesirable to attempt to formulate an official code which would, if inclusive, become too long and too involved. They advised, and the writer believes correctly, that the official code (29)[3] be kept simplified, as at present, with the understanding that the word "client" be considered to be inclusive of the word "employer" which makes the code of much wider and more general application. They advised, however, that someone be appointed by the Board of Direction to prepare an inclusive paper on this subject as a basis for a wide and full discussion and that such paper after discussion be made readily available to all engineers.

The writer was drafted by the Board of Direction to prepare this paper and has attempted, so far as practicable, to cover all ordinary work of the engineer.

In discussing this subject and prior to attempting to suggest rules of conduct for the engineer, it seems necessary to show how important such principles are to success in the engineering profession. It is self-evident that all men desire success in their undertakings; otherwise, they would not attempt them. The question "What is success?" can hardly be answered in any brief or exact way as the ideals of success vary with each individual. The ideals may be centered around wealth, fame, honors, political preferment, professional standing, home and family relations, or many other objectives too numerous to be cataloged. Usually the ideals are composite and include a fair share of several such objectives.

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[2] "The Engineer and His Code," by Daniel W. Mead, *Transactions*, Am. Soc. C. E., Vol. 101 (1936), p. 1483.

[3] Numerals in parentheses, thus: (29), refer to corresponding numbers in the Bibliography; see Appendix.

Whatever may be the ideals of the individual, if they are worthy, their attainment requires health, intelligence, courage, native ability, dependability, education and training, experience and ability to apply theory to practice, character, opportunity, hard work, the will to succeed, and personality. (A good personality is the result of acquiring a wide range of habits and abilities for dealing with other people in the manner in which they like to be dealt with (18).) Each of these attributes is partly independent and partly dependent on individual effort and can be acquired, strengthened, and made effective by the exercise of care, perseverance, and energy.

Every one is born with certain physical and mental endowments and capabilities which should be properly developed. Early development is dependent on home influences and environment but the time normally comes when the individual reaches an age where he must assume responsibility for his own future welfare.

Time is one of the most important possessions of every one and its proper expenditure is very important. If time is wasted in the minor pleasures of life instead of being used in the development and improvement of talents and opportunities then only minor professional advancement can be expected.

To learn so to budget one's time that it may be successfully utilized in acquiring the ability to apply the theory and principles of technology and science to professional practice is the soundest preparation for professional accomplishment and advancement. It should be understood that the engineering student becomes an engineer only when he can apply, successfully, theory to professional practice. A reasonable amount of physical exercise and of social intercourse is both desirable and essential for individual development but excess in either study or pleasure is unfortunate. A well-balanced expenditure of time is the desirable aim. The time spent in the office (usually eight hours per day) in drafting, designing, planning, etc., is too short to assure early acquaintance of the young engineer with his job. Work, rest, and other necessary expenditure of time will commonly leave at least four hours of spare time, and these hours are of great importance in determining the speed of advancement of the young engineer and of his ultimate professional success. The budgeting of these often wasted four hours to average not more than one half to recreation and the other half to study that is collateral to his job will gain one full day of advantage every four days over those whose days end when they pass out of the office door. Early acquaintance with the job and possible early advancement may result to the young engineer through the intelligent use of his spare time.

The mental point of view of the individual is a matter of great importance to his success in life. There is a tendency in the scientific teaching of the present time to consider that the possibilities of the individual's progress are limited and that his advancement is circumscribed by his hereditary endowment, by his intelligence quotient (I.Q.), by his glands, by inhibitions, by inferiority complexes, etc., to a very limited field of endeavor beyond which he cannot attain. In the opinion of the writer, this theory omits the most important factor of the individual—his personal energy and will power. If the individual neglects his own personal power of self-control, and will to achieve, he will float with the tide of affairs and take only that share of life which comes to him naturally. On the other hand, if instead of taking things as they come, he exerts himself through introspection and study, if he determines his own strength, weakness, and defects, and by strenuous personal endeavor attempts to strengthen and improve such weaknesses as he may find in his own character and personality, he can accomplish fine results which otherwise cannot be attained. Defects of character and personality can be overcome by strenuous endeavors of the individual, and by him only. Great accomplishments are possible if the individual has the will to accomplish and is willing to work. Ambition and the will to work toward a desired end will result in high attainments. Genius is but the exercise of infinite pains to accomplish a desired end.

It is very desirable for every man to realize that there are great variations in the normal character and ability of men as well as in the opportunities which occur. These factors necessarily must have a very important effect on their possible development and the degree of their professional success. The higher positions in every line of professional and business life are comparatively few in number when the great number of professional aspirants is considered; and it is only those few who combine high character, great ability, and favorable opportunities who can possibly hope to acquire one of those outstanding positions.

While it is the part of wisdom that each young engineer should recognize that he is probably only an average man, it is well for him to remember also that such positions frequently have been filled by men who have, in early life, occupied the most humble positions. It seems desirable also that each man recognize that careful attention to his own character and progress, and an intelligent and strenuous attempt at the best possible development of himself, will accomplish the most that can be expected under his own individual circumstances.

A strong personal attempt at self-improvement and a personal interest in his work will result not only in the best that each one can hope to accomplish but will assure pleasure and happiness in each man's work, as well as contentment in the position which he will attain regardless of whether or not he attains the heights of ambition to which in his youth he aspired. To do the work which he undertakes in a thoroughly satisfactory manner will usually give more personal pleasure and satisfaction and a greater feeling of personal success and happiness than would a position of greater responsibility and more arduous demands which the individual may be poorly equipped to occupy.

Intelligence and Native Ability.—The writer has always believed that, if one has the ambition and the necessary industry, courage, and determination, any man with fair intelligence and ability can make a reasonable success in any line of work which he desires to enter. The higher the degree of intelligence, the better the chances for greater success, other things being equal. It does not follow that men of the keenest intellect will always make the best engineers, as the ability to apply knowledge and many other factors are of equal importance.

Dependability.—If an investigator considers any engineering position, from that of draftsman in the office or chainman in the field, to the chief or the consulting engineer in the most responsible positions, he will not find one in which dependability is not of equal or greater importance than technical ability and technical skill.

Every engineering enterprise is the combined work of many men and if the best results are to be secured, every one of these men must fulfil his special functions. Weak character is a detriment in all important positions and in all important projects. Every man, ultimately, will be consigned to those places where his character and abilities can be used to the best advantage or where his weaknesses will do the least damage to the project of which he is a part. If he will, every one can train himself to be dependable. Dependability requires not only the necessary technical ability to perform the work on which the individual is employed, but also the characteristics of faithfulness to trusts imposed, honesty of purpose, and loyalty to the service in which the individual is engaged.

The dependable man is assured employment as long as opportunity for his services exists, but the nondependable employee will be discharged as soon as he can be replaced, conveniently, by one in whom dependence can be placed. Lack of dependability and failure to "carry on" in accordance with instructions renders the individual useless in engineering or other business relations.

Technical ability may increase with experience, practice, study, and effort; but lack of character is a fatal defect which, unless remedied, will surely result in failure.

Technical Training and Technical Ability.—It is a truism that, to fill successfully any position, any man must have enough technical ability and technical knowledge and training to perform properly the functions of the position he is to hold. In addition, if he is to attain advancement and a more important position, his knowledge and training must be extended and increased so that he may be known to be capable of meeting the higher requirements of such a position successfully. For this reason, the college commencement is not the end, but the beginning, of strenuous professional study.

Technical education and technical skill, however, are never rated as more than 30% of the attributes required for maximum success; the other 70% is made up of character, personality, and other moral and mental attributes.

The necessary training and education are not confined to work in an engineering school. Many of the most successful engineers of the past have not had the opportunity for college work. In some way they have had the opportunity to become associated with some form of engineering work and their interest and ambition have become so aroused, and they have worked and studied to such advantage, that many of them have achieved a marked degree of success in the engineering profession. Men with ambition, fair ability, and determination will always find success possible in the profession, with or without the benefits of a college course.

In general, at the present time, the most common method of entering the engineering profession is through the engineering schools. Most students of engineering education will agree that the most important subjects in the curriculum are mathematics and the natural sciences and that the study of professional subjects in any great detail is inadvisable, because all such subjects are changing rapidly and few schools can keep in step with those changes. However, a certain amount of professional study is advisable in engineering education for, through this means it is possible to give the student some idea of the application of theory to practice.

Ability to Apply Principles to Practice.—The ability to apply knowledge to the practical affairs of the profession is a most important matter and one that the college man must acquire if he is to be successful in his work. The practical man who studied as he worked has comparatively little trouble in this respect as his studies have been for a particular purpose. For this reason the writer believes that some knowledge of practical things is of great importance in orienting the studies of the school. When a student has had practical experience, he usually has a better idea of the purpose of his studies and thereafter he often pursues them with much greater vigor and profit to himself for he then recognizes their utility.

Occasionally men of high intellectual attainment, who have been in school continuously, with no contact with practical life, have great difficulty in applying their knowledge to engineering work and therefore fail to attain the degree of success which their intellect would seem to warrant.

The writer believes that the opinions expressed herein are essentially the opinions of most mature men in the profession. The evidence of this is best shown by the investigation of Prof. C. R. Mann,[4] M. Am. Soc. C. E., which was made for the Carnegie Foundation for the Advancement of Teaching. In the spring of 1915 letters were sent to the members of the national technical societies requesting individual opinion on the essential characteristics of an engineer. A report[5] was made in the spring of 1916 as a result of the inquiry, in which the essential factors were summarized into six groups which were numbered in the order of their importance as determined by the frequency of their occurrence in about 1,500 replies. Later a new letter was mailed to the members of the societies asking for further consideration of the six characteristic groups previously outlined. To this letter, 5,441 replies were received which were sufficiently definite for classification. From these letters the most probable values of these groups of qualities were computed with the results shown in Table 1, which seemed to indicate a rather definite ideal of the professional engineer as to the characteristics necessary for the greatest success.

It will be understood that the relative weight assigned to the groups of personal characteristics in Table 1 indicates in a general way only the comparative values of the desirable personal characteristics of successful engineers as expressed by the members of the national engineering societies who answered the letter. All of these characteristics are necessary for success, and any characteristic, like any link in a chain, must be able to stand any strain to which it may be subjected or the character, like the chain under similar circumstances, is useless and hence a failure. Every engineer must be capable of performing, satisfactorily, in the positions he is called upon to serve, and if he lacks the strength of character necessary for such positions it is fatal to his success.

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[4] Address on "Engineering Education," by C. R. Mann, *Proceedings*, Am. Soc. C. E., February, 1916, p 98.

[5] "What is an Engineer," by C. R. Mann, *Engineering Record*, Vol. 74. 1916, p. 10.

The important matter is the great weight given to characteristics on which dependability and integrity rest as compared with those factors common to educational requirements; and yet those latter factors are absolutely essential to the success of the engineer.

TABLE 1.—CHARACTERISTICS NECESSARY TO SUCCESS IN ENGINEERING

Item No.	Characteristics	Estimate (in Percentages), Based on:	
		1,500 replies	5,441 replies
1	Character, including integrity, responsibility, resourcefulness, and initiative	41.0	24.0
2	Judgment, including common sense, scientific attitude, and perspective of life	17.5	19.5
3	Efficiency, including thoroughness, accuracy, and industry	14.5	16.5
4	Understanding of men, executive ability	14.0	15.0
	Sub-total	(87.0)	(75.0)
5	Knowledge of fundamentals	7.0	15.0
6	Technique of practice and of business	6.0	10.0
	Sub-total	(13.0)	(25.0)
	Total	100.0	100.0

It is evident, therefore, that, in the opinion of experienced engineers, every successful engineer must be a man of integrity and dependability as well as a man of ability. This does not mean that a man cannot run a transit or level, or be a good draftsman even if he is dishonest, but it does mean that such a man cannot occupy, successfully, a high position of trust and responsibility.

Much could be written covering the desirability and the necessity of high ideals, proper conduct, and courtesy in engineering relations, but the writer believes that enough has already been written to bring these facts "home" to any thoughtful man.

Briefly, it follows that, for success, the practicing engineer must have the characteristics of dependability, stability, perseverance, honesty, thoroughness, courage, foresight, enthusiasm, and cooperation; and for the greatest success there must be included a good character, a fine personality, cheerfulness, loyalty, courtesy, tact, care, economy, sincerity, and harmony. These qualities, together with opportunity, are fundamental to the highest engineering success.

Opportunity.—The best engineering student, if located on a desert island, would never be able to make a success in the engineering profession. Opportunity must obtain if any man is to succeed. In normal times many opportunities are open to the engineering student and usually after the opportunity obtains the question of success depends upon the individual "making good" on his first job which is commonly a stepping stone to a better position and to wider opportunities.

Opportunities are sometimes available through relatives, friends, and acquaintances, but as a rule the man must be able to fill the position satisfactorily or he cannot retain it. In applying for a job, good manners, frankness, modesty, and a good personality are favorable factors.

Success (however it may be defined) depends on two essential factors—opportunity and the man. Rarely, success may be phenomenal. A man especially qualified and, perhaps it may be said, especially inspired, seizes an opportunity and meets it so successfully that he arouses public approbation and gains fame, honors, and many additional opportunities which apparently afford an open way to fame and fortune. Such opportunities, however, are the exception and not the rule. Most successful men acquire success only by the long hard road of individual effort accompanied, of course, by utilizing the conditions that obtain by the intelligent application of their education, training, and experience. Most men have attained success in the past and will attain success in the future, in this longer and more arduous manner.

The common idea conveyed by the well-known verses of John J. Ingalls, that opportunity knocks once and only once at every door and if unheeded departs and never returns, is fallacious and pessimistic. Opportunity is almost always present and almost always at hand if the individual has the intelligence to recognize it and the energy to utilize it. The discouraging feature to the young is that, commonly, opportunity presents itself in the disguise of hard work and, therefore, is not always recognized nor received with enthusiasm.

The actual conditions are best stated in the following poem entitled "Opportunity," by Walter Malone:

They do me wrong who say I come no more
  When once I knock and fail to find you in;
For every day I stand outside your door
  And bid you wake, and rise to fight and win.
Wail not for precious chances passed away!
  Weep not for golden ages on the wane!
Each night I burn the records of the day—
  At sunrise every soul is born again!
Dost thou behold thy lost youth all aghast?
  Dost reel from righteous Retribution's blow?
Then turn from blotted archives of the past
  And find the future's pages white as snow.
Art thou a mourner? Rise thee from thy spell;
  Art thou a sinner? Sins may be forgiven;
Each morning gives thee wings to flee from hell,
  Each night a star to guide thy feet to heaven.
Laugh like a boy at splendors that have sped,
  To vanished joys be blind and deaf and dumb;
My judgments seal the dead past with its dead,
  But never bind a moment yet to come.
Though deep in mire, wring not your hands and weep;
  I lend my arm to all who say "I can!"
No shame-faced outcast ever sank so deep
  But yet might rise and be again a man!

CODES OF CONDUCT AND ETHICS

In preparing the codes of conduct and ethics which follow, the writer has drawn freely from the various codes already adopted by other professions and industries and has included only a few matters which have not been included in other codes. As many matters included in this paper are embodied in many codes, he has not been able to give the original source of each particular principle. In general, each ethical principle is presented without argument as most of them, it is believed, will be accepted as axiomatic. The validity of any principle, questioned by any member of the profession, may be raised in the discussion, which it is hoped will be very general and complete.

REQUIREMENTS OF THE PROFESSION

The profession of engineering calls for men with honor, integrity, technical ability, business capacity, and pleasing personalities. The engineer is entrusted with investigating and reporting on the advisability and feasibility of public, semi-public, and commercial works. On the basis of his reports, important works are inaugurated in which large expenditures are involved, and securities sold and purchased. Often the limited means of widows, orphans, and other dependent persons are invested on the basis of his knowledge and good faith, and the results of financial failure of such projects are very serious. The engineer is also entrusted with designing and constructing important public and commercial undertakings in which his honesty of purpose and sincerity of statement must be above suspicion. He acts as a professional adviser, and his advice must be honest and disinterested; he must exercise judicial functions between his clients and contractors, manufacturers, and material supply companies, and his decisions must be fair and impartial. He has moral responsibilities to the public and to his superiors, to his associates and subordinates, which should be exercised with the highest ideals. His work carries with it grave responsibilities to the public which demand his conscientious consideration; and he cannot properly perform his numerous functions unless his ability, conduct, and motives are such as to command the highest respect and confidence.

PRINCIPLES APPLICABLE TO ALL PROFESSIONAL POSITIONS

It may appear to the younger members of the profession that such requirements apply only to the heads of the profession, but it must be remembered that the profession is not static. The business and engineering world is full of examples of men who have started in the humblest positions and who by means of the proper development of their knowledge, intelligence, character, personality, and ability have become the heads of industry and of the professions in the United States. Opportunities are open to every man who has the personality, initiative, ability, and determination to strive for better things; and the hope of advancement should spur every young man to fit himself for the higher positions which soon will become available for new men, as the retirement of those of advanced age, or as changed conditions, make necessary any advancement in the personnel of each industry. The younger man should recognize that his future is involved and he should prepare himself for those higher responsibilities.

The writer has not attempted to do more, in general, than to state ethical principles as this presentation is intended to offer a basis for what it is hoped will entail discussions, by members of the profession, of this very important subject.

Although the following principles are separate under different headings, it should be understood that these principles should be used by all engineers so far as applicable in all their professional relations.

I. The Engineering Student and the Young Engineer

The engineering student should understand that he is preparing to enter a profession in which success, in general, is the reward of honest and strenuous effort. Professional success, while always involving opportunity, usually depends largely on the individual himself and on his intelligence, his personality, his ambitions, his courage, his industry, and his determination.

1. The mental discipline necessary to do well the work of a required but distasteful course is of great value, as throughout life one must frequently solve new problems and do things one would prefer not to do. Hence, if the student finds his required course includes subjects which he does not like, nevertheless he should give such subjects sufficient attention, for the ability to do work well whether he likes it or not is an essential factor to success.

2. It should be remembered that the college commencement is not the end of professional study but rather that it is the beginning of special investigation and the acquisition of knowledge of the practical application of the fundamental principles acquired in college. If a man is to succeed in any branch of the profession he must keep abreast of all new developments in that special field. He must be a constant reader of all books and periodicals that deal with the subject. He must (through active participation in the work of technical societies) keep in touch with the men who are doing important work in engineering. In brief, he must continue to be a student for the remainder of his life.

3. The engineer's employers will seldom ask him what he likes to do for it is his business to like the duties assigned to him. Interest in the job at hand will induce study and investigation as to how it can be best and most economically designed and completed. Only when the engineer feels an interest in his work can be accomplish the best results.

4. Even if the young engineer's immediate work is foreign to that which he intends to follow in later life, a thorough study and understanding of any engineering subject will be found of great advantage in his later years.

5. The last half century has witnessed a great development in engineering literature. This is contained in engineering society publications, in engineering magazines, and in engineering books with which few young engineers are familiar. A knowledge of this literature, especially along the line of the young engineer's specialty, is of great importance. By extensive study the young engineer should utilize this great storehouse of information. "The difference between an educated and an uneducated man is not so much in the greater amount that the educated man knows as in his ability to find out"—Huxley.

Every one must build largely on the experience and precedents of the past. The young engineer should not be rigorously ruled by precedent, however, but he should not ignore it. On the other hand, he should not strive too much for originality. That which is entirely new is likely to be bad in principle or design. When, after thorough study and investigation, a new principle, method, or design is found to be more desirable, the engineer should not hesitate to adopt it; otherwise no improvements in principle, practice, or methods would be possible.

6. Frequently new ideas and new plans are outlined in more or less detail in the engineering press. Such plans seldom should be adopted without careful inquiry as to their success after some years of practical use. Often, apparently good and economical methods or design prove unsatisfactory in practice because their success or failure frequently can be determined only by use. Where new or untried methods are contemplated, the site of such works, preferably, should be visited and inquiry made as to the degree of success which has been encountered in actual usage before such plans are adopted.

7. The young engineer owes it to himself to join such local and national technical societies as are pertinent to the work in which he is engaged; he should attend their meetings whenever practicable, should read the papers at least in so far as they refer to his line of work, present or prospective, and should join in the discussions verbally or in writing whenever he has information which is new or important. Such association results in new friends and acquaintances, and such participation shows his interest in and knowledge of the work and, when new work is proposed, his availability for such future work. Many successful engineers owe their prominence in no small degree to their association with, and participation in, these engineering societies.

8. The young engineer should not take himself or others too seriously; even if he has been near the head of his class, he will find that the application of principles in practice is often a difficult matter, and requires study, inquiry, and sound common sense. The older members of the profession, men of long experience and extensive practice, should be recognized as human and fallible. Their opinions should be taken subject always to careful consideration, and should not be accepted without careful thought as to their application to the problems at hand. The young engineer should not be a "yes" man; he should agree only when, after careful study, he can conscientiously adopt, as his own, the opinions expressed by others.

9. Some older engineers depend too much upon past experience and attempt to solve, too quickly, new problems on the basis of such experience, and without careful consideration and study. Every problem is a new problem involving some new conditions, and hence requiring careful investigation and thorough study. Any engineer who will not give a new problem due consideration and study should be retired and thus give the man who will do so an opportunity.

10. If possible, every engineer should visit the plants on which he has assisted in the design and construction after it has been in operation for a few years. Preferably, he should visit them without the announcement of his connection with their origin; and he should inquire of the operators as to any faults or troubles in stability, safety, and operation. If the engineer appears as a stranger, he is likely to obtain information which may improve his future

designs, although it may not always add to his self-esteem. Many engineers, for lack of contact with the work they have already done, continue for years making the same mistakes in design.

11. The young engineer should do his work as if he were personally responsible for the success of the entire project on which he is engaged. He should do his work as if it were his own and as if his own funds were involved. The personal point of view should always be the basis on which the engineer's work is done. "Will it be safe and economically sound?" "Would I do this work at all, and if so would I do it this way if it were mine?" The questions should always be in the mind of the engineer in charge of any project. It is not always practicable for the engineer, if in a subordinate position, to influence a design so that it will accord with his ideas of economic and sound practice but his study of the fundamental principles and their applicability is essential to his future success.

12. A pleasing personality is of the highest importance. Cultivate, so far as possible, optimistic tendencies; look at the pleasant and attractive side of life; cultivate a friendly attitude toward those whom you meet, and meet them with a smile. They will respond, generally, in a similar manner. This will smooth the way for the adjustment of differences in opinion and the reconciliation of misunderstandings.

13. Consideration for, and a real interest in, others is the foundation of a fine personality, and when accompanied by ability is the surest way to success. In some cases the rough character, heedless of the feelings or the rights of others, may succeed by sheer ability and in spite of—not on account of—his unfortunate personality.

14. The young engineer searching for a job should seek employment under men of recognized standing and ability and of high ideals. Association with men of low ethical standards is sure to affect, seriously, the mental attitude of the young man who is likely to assume such standards are common to the profession. It is much better to serve with a man of fine reputation and fine character even at a financial sacrifice.

15. Truthfulness and dependability are the fundamental basis of all proper conduct. The engineer should agree to accept a position only after he has fully made up his mind to carry out his agreement. His failure to do so on account of a later offer at a higher salary is unethical and therefore a very unsatisfactory way to begin work in the profession.

II. A Code of Courtesy and Personal Conduct

A code of courtesy and personal conduct is expressed in the following items (25):

1. The engineer should not take himself or others too seriously. All men are human and subject to error.

2. He should cultivate a sense of humor. It brightens life and eases intercourse. Serious matters, however, should always receive serious consideration and in such affairs levity is out of place.

3. He should not fail to smile and use courtesy to all—equals and subordinates as well as superiors. This includes greetings, tone of voice, and consideration for the personal comfort of all who enter his office, whether visitors or employees.

4. He should not criticize any one adversely in the presence of others.

5. He should not use ungentlemanly language.

6. He should not fail to respect the authority of others, and the personal dignity of subordinates.

7. He should not criticize destructively. He should criticize constructively by suggesting how the error in question may be avoided in future.

8. He should not encourage gossip about fellow workers.

9. He should not countenance animosities or intrigues.

10. He should not fail to give every one credit for good suggestions.

11. He should not fail to be as liberal in praising good performance as he is in censuring bad performance.

12. He should not show partiality or favoritism or injustice in any direction.

13. He should not let personal feelings govern his actions against his better business judgment.

14. He should not belittle any one, not even the humblest worker with whom he is associated.

15. He should not usurp the functions of others or "go over their heads," or undermine the standing of another, or carry tales.

16. He should not carry personal dislike into organization work; if he does not like an associate, he should subordinate his feeling to the general good of the organization.

17. He should not fail to be generous if he finds himself in a position to criticize or upheave another department or individual—the turn of the wheel of business fortune may place him in a similar position.

18. He should not fail, wherever possible, to give the benefit of his thought and information to others in an organization with which he is connected.

19. He should not fail, if he must disagree radically with an associate, or must "fight" him in an organization, to announce the fact frankly to him first, and fight clean and with good nature.

20. He should not fail, if he must resign from an organization, to do so with due regard for his associates whose interests may be affected by his leaving.

21. He should not use merely technical advantage over an associate in an organization. He should guide himself by sound business principles and the spirit of fairness, even under provocation.

22. He should not take advantage of double meanings of words, or unsaid things or unknown facts, or hide behind sophistries or cryptic statements. He should cultivate clearness, letting all know precisely where he stands. Good strategy in business does not mean stealth and secrets; speed, analysis, and publicity are better.

23. He should not make a move involving his associates without first consulting them.

24. He should use correct but simple English in speaking and writing. Proper language is the mark of education, cultivation, and accuracy.

25. When work is to be done, he should not delay. Putting off to the future the things which should be done at once is one of the habits that leads to unsatisfactory service and to failure.

III. PUBLIC RELATIONS OF THE ENGINEER

It is fundamental that all principles for correct human action and conduct, to be of permanent or even of reasonably lasting character, must be for the best interests of the public; and, although a transitory advantage to a class, a trade, a profession, or an individual may sometimes temporarily be gained by unwise, unethical, political, unlawful, or even by criminal means, such advantage can be only short lived. Ultimately, it will result in a reaction detrimental to the final welfare of the class, trade, profession, or the individual who stoops to the use of such unfair and unethical methods.

1. To do unto others as he would that others should do unto him (26) is the basis of all sound ethical action, and this principle should be applied not only to the immediate personal relations of the individual, but also to the broader relations as they affect associates and the public.

2. The laws of the state and the nation are paramount, and every engineer is bound to support them in principle and practice, and in all reasonable ways to encourage their enforcement.

3. It is the duty of the engineer to satisfy himself to the best of his ability that the enterprises with which he becomes identified are of legitimate character. If, after becoming associated with an enterprise, he finds it is of unsound or questionable character, he should sever his connection with it as soon as practicable. The engineer should engage in no occupation nor undertake any project that is contrary to law or which is inimical to the public welfare.

4. The engineer should endeavor to assist the public and public officials to a fair and correct understanding of engineering matters, to extend the general knowledge of engineering, and to discourage the appearance of untrue, unfair, or exaggerated statements on engineering subjects in the press or elsewhere, especially if those statements may lead to, or are made for the purpose of, inducing the public to participate in unworthy enterprises.

5. In all his actions, the engineer should avoid the appearance of evil; he should not, by word or deed, indicate that he is willing to compromise with either legal or moral standards.

6. The engineer should discourage, in every legitimate manner, the construction of public works that are economically unsound for the community, state, or nation. As engineers in public service are barred, in general, from a frank public expression of opinion concerning such projects, those who are free from the obligations of employment of public bodies are especially obligated to undertake such discussion.

7. Technical discussions and criticisms of engineering subjects should seldom be conducted in the public press but, when possible, before engineering societies or in the technical press. Clear and simple statements of facts and

principles that will clarify local problems in the public mind may sometimes be both desirable and essential for the public welfare and for information of the public on local technical matters.

8. It is unprofessional to give an opinion on a subject without being fully informed as to all the essential facts relating thereto and as to the purposes for which information is asked. The opinions should contain a full statement of the condition under which such opinion applies.

9. The engineer in charge of public work or in private work accessible to the public should be mindful of the safety and the convenience of the public. He should use kindness and courtesy in his personal contacts with the public and should furnish such information to inquirers as is consistent with the interest of his employers and reasonably possible without undue interference with his professional duties.

10. It is unethical to give expert testimony or to advise his client in a manner differing in substance or implication from what he would present and defend before a meeting of this Society.

11. It is unethical to submit a proposal, or to enter into contracts, for the construction of work, for which plans and specifications have been prepared by him as engineer for a client.

12. It is unethical for an engineer to be associated in the conduct of professional work with others who do not conform to the standards of this code.

IV. The Personal Relations of the Engineer

To do unto others as he would have them do unto him (26) is the basis of all sound rules of action.

1. To attempt to injure, falsely or maliciously, the prospects, position, or business of another is unethical and unworthy of any reputable engineer.

2. Honorable competition for promotion and opportunity for employment is an essential part of modern democratic civilization; but an attempt to supplant another after definite steps have been taken toward his employment is dishonorable.

3. It is dishonorable to undertake professional work at the price that will not permit the work to be designed properly, supervised thoroughly, and completed satisfactorily.

4. It is unethical for an engineer to compete with another engineer for employment on the basis of professional charges by reducing his usual charges and in this manner attempt to underbid after being informed of the charges named by another.

5. It is unprofessional to accept an appointment while the just claim for compensation or damages—or both—of an engineer previously employed remains unpaid or a settlement is agreed upon unless he or his heirs or assigns neglect to press his claims legally within a reasonable period.

6. The use by an engineer of plans and specifications prepared especially for him by manufacturers, supply houses, or patentees, in order to reduce his work and his charges, and thus unfairly compete with other engineers in professional employment, is unfair and unethical.

7. It is unethical for an engineer to create discord among the clients or customers of any individual or corporation furnishing any kind of public or private service, in order to secure the opportunity of furnishing plans for similar service on the pleas of greater satisfaction or reduced cost. This statement should not be considered to preclude an honest report by an engineer employed, without personal solicitation, to report on the possibilities of betterment in public or private service.

8. It is unethical to review the work of an engineer for the same client, except with the knowledge or consent of such engineer, without giving such engineer an opportunity to present his reasons for the plans in question, unless the connection of such engineer with the work has been terminated; and even in that event, the engineer who has prepared the plans under consideration should have the opportunity to explain and defend them.

9. Rule 8 should not be understood to prevent the investigation, with or without notification to the engineer, of any engineering work for which purchase or refinancing is contemplated by another person or persons—or, on work in which improper design or improper or dishonest work is suspected—when such investigation is made for other than the same client or when the investigation is made for parties having a legitimate interest in the economy, safety, and honesty with which such works are constructed, or where such investigation is made in the public interest.

10. It is dishonorable to advertise in self-laudatory language or in any other manner derogatory to the dignity of the profession.

11. It is dishonorable for any engineer to use the advantages of a salaried position, public or private, to compete unfairly with other engineers.

12. It is unprofessional for an engineer to have a direct or indirect interest in any business which will in any way prejudice his judgment in regard to any professional work which he may be called upon to perform or on which he may be engaged.

13. The engineer should take an interest in, and should assist, his fellow engineers by exchange of general information and experience, by instruction and similar aid, through the engineering societies or by other means. He should endeavor to protect all reputable engineers from misrepresentation.

14. The engineer should take care that credit for engineering works or engineering designs is attributed to those who, so far as his knowledge of all matters goes, are the real authors of such work.

15. An engineer in responsible charge of work should not permit nontechnical persons to overrule his engineering judgments on purely engineering subjects.

16. When any question of design or construction becomes essential to the safety or to the success of a project the engineer should insist on the correct procedure even if a refusal must result in his resignation. During his fifty-six years of practice, the writer has had occasion several times to lay his job "on the table" before his clients, once before the governor of his state, and at other times before city councils or their committees, boards of directors of corporations, and private clients with the general statement: "This is your job and you are entitled to have it done any way that you see fit; but you cannot, for the

reasons that have been stated, have it done in the way you propose as long as the speaker is your engineer." The question naturally arises, "How often has the writer lost his job on account of taking such a position?" The answer is "Not once." The writer is not advising unnecessary arbitrary dictation as to non-essential details. He would paint his client's house black if after due consideration his client so desired; but he would not knowingly build a poor or unsafe foundation under the house for the sake of any job.

17. It is unprofessional for an engineer to undertake professional work for which he is not qualified by education, training, and experience. This statement should not be understood to preclude engineering students accepting any position offered to them by those who understand the limits of their education and practical experience.

18. As a public official the engineer should realize his own limitations in both training and experience, and should not hesitate to ask and advise consultation on unusual problems on which his own information or experience is incomplete. Elective and also appointive officials often assume that they are endowed with all information and judgment necessary for the fulfilment of their function, and engineers—being human—are not free from this serious defect the result of which is often unnecessary extravagance and wastefulness.

19. Professional work should come to the engineer in general practice without solicitation and on the basis of experience and reputation, as in the case of the medical and legal professions. This is the end to be sought. Honorable competition is often necessary under present conditions, but solicitation by lobbying, the use of influence, criticism of competitors, self-laudation, and other sale-agent practices are degrading to the individual and to the profession. Competition by bidding for engineering work should be discouraged. When municipalities receive propositions for engineering work the engineer, if he submits a proposition at all, should do so only on the basis of his regular charges. The offer or payment of commission to secure work, either directly or through an agent, is unethical. It may be illegal and possibly criminal. In general, he should accept no favors that he cannot and does not return in full measure.

20. The conscientious personal point of view of the engineer should always be the basis of his advice in matters of safety in structures, in plans, or in investments; he should always ask himself before he advises a client: "If I were making an investment, would I invest my money in this project?" "If I were building this structure for myself, would I make it any different or any stronger or safer?" "Have I included in my design unnecessary expenses or extravagances?"

If the engineering advice is in accordance with an honest answer to these questions, it is the best advice that the engineer can give; and if it is in error it is an honest error.

V. The Engineer's Relations to Client or Employer

1. The engineer should consider the protection of a client's or an employer's interests, in all honorable ways, his first professional obligation, and, therefore, should avoid every act contrary to this duty. If any other considerations, such as personal or professional obligations or restrictions, may interfere with his

meeting the legitimate expectation of a client or employer, the engineer should inform the client or employer of the situation before employment is accepted or when such conditions arise. This applies equally to the relations of all engineering employees and employers (private clients, corporations, consultants, or government bodies), regardless of salaries received and as long as employment is retained.

2. The engineer should act for his clients or employers only as a faithful trustee or agent, and should accept no remuneration for his work other than his stated salary or charges for services rendered. The engineer should accept no favors, and no collateral employment that might tend to prejudice his position or modify his judgment in the performance of his duties to his clients or his employer.

3. It is unprofessional for an engineer to accept any commission, remuneration, or substantial service or benefits from a contractor, a manufacturer, or from any interested party other than his client or employer. The rejection of favors, or offers of employment, however, should be made pleasantly and without offense. The acceptance of minor courtesies common among friends and acquaintances, such as cigars, theater tickets, meals, etc., may occasionally be sanctioned, but such courtesies should be returned in kind and, as a rule, should leave the credit on the side of the engineer.

4. In accepting service in a professional capacity, unless otherwise specifically agreed, the engineer's implied legal responsibility warrants the assumptions: That he has the required skill and knowledge; that he will use reasonable care and diligence in the exercise of his function; that he will use his best judgment; and that he will be honest and faithful to his trust. It is unprofessional, therefore, for an engineer to guarantee his estimate or contract by bonds or otherwise.

5. The engineer should refuse to undertake work which he believes will result in failure of purpose, or which will be unprofitable to a client, without first advising his client as to the nondependability of successful results.

6. It is inadvisable to accept service on questionable projects even when the client is so advised, as the engineer's connection with such a project will undoubtedly cause his name to be connected with a partial or complete failure and may induce investments in the project by others who may believe that his connection with it is an endorsement of its success.

7. Whenever an engineer is employed by a client without definite agreement as to his compensation, he is not warranted in charging more for his services than the fee which he would have charged had such fee been discussed in advance of his employment. It is wise for the engineer to consider carefully the real value of the work he has done for his client and reduce his bill if he finds his charges are higher than are warranted by the real value of the services rendered.

8. An engineer called upon to decide the use of inventions, apparatus, or anything in which he has a financial interest, should make his status in the matter clearly understood by his client or employer before engagement.

9. An engineer in independent practice may be employed by more than one party when the interests of the several parties do not conflict; and it should

be understood by all interested that he is not expected to devote his entire time to the work of one, but is free to accept other engagements. A consulting engineer, permanently retained by a party, should notify others of the affiliation before entering into relations with them, if, in his opinion, the interests might in any way conflict.

10. An engineer should consider it his duty to make every effort to remedy dangerous defects in apparatus or structures or dangerous conditions of operation, and should bring such conditions to the attention of his client or employer.

11. When an engineer undertakes, for others, work in connection with which he may make improvements, inventions, plans, designs, or other records, he should preferably enter into a written agreement with regard to their ownership. In general however (27),

A. A client does not acquire an exclusive right to plans or apparatus made or constructed by a consulting engineer except for the specific case for which they were made.

B. If an engineer uses information which is not common knowledge or public property, but which he obtains from a client or employer, the results in the form of plans, designs, or other records shall not be regarded as his property but as the property of his client or employer.

C. If an engineer uses only his own knowledge or information or data, which by prior publication or otherwise are public property, and obtains no such data from a client or employer except performance specifications or routine information, then the results in the form of inventions, plans, designs, or other records should be regarded as the property of the engineer, and client or employer should be entitled to their use only in the case for which the engineer was retained.

D. All work and results accomplished by the engineer in the form of inventions, plans, designs, or other records, or outside the field for which a client or employer has retained him, should be regarded as the engineer's property.

E. Engineering data or information which an engineer obtains from his client or employer, or from other engineers, or which he creates as a result of such information must be considered confidential by the engineer; and although he is justified in using such data or information in his own practice as forming part of his professional experience, its publication without express permission is improper.

F. Designs, data, records, and notes made by an employee, and referring to his employer's work, should be regarded as his employer's property.

12. The engineer should regard every position as a part of the foundation for his future, with the understanding that each position (even if it is poorly paid) will be the forerunner of better things if it is filled well and intelligently. A reputation for high ideals, good personality, honesty, dependability, intelligence, initiative, and professional ability is the foundation of professional preferment.

13. It is unethical for an employee while retaining his position with an employer to solicit or to accept other employment to investigate, design, or to construct work in competition directly or indirectly with his employer or which will engage his time, energies, and interest to the detriment of the regular work for which he is receiving compensation.

14. No employee is warranted in doing, directly or indirectly, any dishonest or illegal thing although such action may be required or expected by the employer. With his principles and his future in mind, he should refuse to take part in any dishonest act and should be ready to quit his job if necessary. When questionable actions or tendencies are evident on the part of his employer, the employee should seek other employment immediately rather than retain his position and be slowly but surely influenced to ignore good ethics and ultimately to comply with unethical or possibly dishonest practices.

15. The employee should begin his work promptly on time and continue it until the regular hour for quitting work. The employee who is faithful to his task is always recognized although this recognition may not always be apparent. The employee who is usually late in arriving and early in departing from his work will likely be the first to be discharged when work is slack.

VI. THE ENGINEER'S RELATIONS WITH HIS EMPLOYEES

1. The chief engineer or an engineer employing others in the profession owes to his assistants or employees proper consideration of their personal, financial, and professional welfare and advancement.

2. He should encourage their professional study and initiative, and also high ideals of personal and professional conduct.

3. He should criticize, kindly and sparingly, and commend freely any evidence of good judgment, good suggestions, good work, and good professional conduct.

4. He should be free with professional advice and information, and should encourage his assistants in professional improvement.

5. He should so conduct himself as to command their regard and respect for him both as a man and as an engineer.

6. In his office he should be pleasant and courteous to all subordinates, equals, or visitors in greeting, conversation, and in consideration for personal comfort of all who enter.

7. Correct English should always be used and ungentlemanly language studiously avoided.

8. Other things being equal, promotion should be made on merit only, and preference should be given to employees over outsiders.

9. Partiality, favoritism, and injustice should be avoided studiously.

10. Even the most humble worker in an organization should never be belittled. No one should be criticized adversely before others. The authority of others and the personal dignity of subordinates should be respected.

11. Where criticism is necessary it should be constructive, and the employees should be shown how the error in question can be avoided in the future.

12. He should not fail to give every one credit for good suggestions.

13. When an employee is entrusted with a design, his ideas should be utilized so far as practicable. No design under development should be changed arbitrarily or without sound reasons. Only by reasonable treatment of this kind can the initiative of the office force be retained and its greatest value utilized.

14. If necessary to discharge or deprive workers of authority, he should not fail to do so on the basis of business expediency rather than on personal failure.

15. He should not fail to accept blame, from an outsider, for the mistakes of his organization, and should not "air" inside grievances or differences of opinion.

16. He should not "fire" an employee, but should part with him with regret, delicacy, and personal good wishes; on strict organization principles the employer engineer is to blame for all failure among subordinates.

17. The use of the word "we" instead of "I" in discussing the engineering accomplishments of his office is highly desirable.

VII. Relations of the Engineer with the Contractor

The engineer is not only employed to design and prepare plans and specifications for public and private works, but he is commonly also made arbiter between his client or employer and the contractor who is to furnish materials, machinery, equipment, and labor for the construction of such works. In such position there is sometimes a tendency on the part of the engineer to feel that his principal duty is to his client or employer and too often this feeling results in more or less unfair treatment of the contractor.

1. It is unwise for the engineer to ask contractors to prepare preliminary estimates of cost without reasonable compensation for the time and expense of such estimates. The contractor's knowledge is due to his experience and study, and he should be compensated for his services in the same manner as the engineer would expect to be compensated under similar conditions.

2. In his judicial position as arbiter or umpire between his client or employer and the contractors doing work or furnishing materials or equipment for the works he has in charge, the engineer should act with the highest ideals of fairness and honor.

3. Plans and specifications should be prepared as definite and as specific as possible and should define the exact requirements clearly in every case. They should admit of no double meaning.

4. It is unethical to issue specifications which are unfair or so indefinite that they may be interpreted favorably or unfavorably according to the individual contractor who may be awarded the work.

5. Bids should not be opened until the official date of the letting. Lettings should be conducted with fairness and openness; and, in general, all bids should be opened and read publicly in the presence of all bidders who desire to be present.

6. No change in bids should be allowed after bids are opened and read, and the work should be let to the lowest responsible qualified bidder.

7. In private work no contractor to whom the contract will be refused, if he is the low bidder, should be invited or allowed to bid on the work.

8. The engineer should neither use nor countenance the use of the low bid to secure a reduction in the bidding price of some favored competitor.

9. No work should be advertised for bidding until definite and complete arrangements have been made for financing; otherwise contractors may be put to unjust and unnecessary expense.

10. It is unethical for the engineer to require the contractor, without just compensation, to do work or to furnish material not clearly covered by the plans and specifications, but which may be required to complete the work or to cover defects in the design, plans, or specifications.

11. In matters of dispute, the engineer should always render such decisions as he would feel were justified if he were: (*A*) The owner or employer, or (*B*) the contractor.

VIII. Business Relations

Engineering is not only a profession, but in many cases is related closely to business. In consequence, the ethics of business should be observed by the engineer in all business relations. The business principles enumerated by the U. S. Chamber of Commerce are therefore applicable to the engineer in all his business relations (30):

1. The foundation of business is confidence, which springs from integrity, fair dealing, efficient service, and mutual benefit.

2. The reward of business for service rendered is a fair profit plus a safe reserve commensurate with the risks involved and foresight exercised.

3. Equitable consideration is due in business alike to capital, management, employees, and the public.

4. Knowledge, thorough and specific, and unceasing study of the facts and courses affecting a business enterprise, are essential to a lasting individual success and to efficient service to the public.

5. Permanency and continuity of service are basic aims of business, so that knowledge gained may be fully utilized, confidence established, and efficiency increased.

6. Obligations to itself and society prompt business to strive unceasingly toward continuity of operation, bettering conditions of employment, and increasing the efficiency and opportunities of individual employees.

7. Contracts and undertakings, written or oral, are to be performed in letter and in spirit. Changed conditions do not justify their cancelation without mutual consent.

8. Representation of goods and service should be truthfully made and scrupulously fulfilled.

9. Waste in any form—of capital, labor, services, materials, or natural resources—is intolerable, and constant effort should be made toward its elimination.

10. Excesses of every nature—inflation of credit, over-expansion, over-buying, over-stimulation for sales—which create artificial conditions and produce crises and depressions are condemned.

11. Unfair competition—embracing all acts characterized by bad faith, deception, fraud, or oppression, including commercial bribery—is wasteful,

despicable, and a public wrong. Business will rely for success on the excellence of its own service.

12. Controversies will be adjusted, where possible, by voluntary agreement or impartial arbitration.

13. Corporate forms do not absolve from or alter the moral obligations of individuals. Responsibility will be as courageously discharged by those acting in representative capacities as when acting for themselves.

14. Lawful cooperation among business men and in useful business organizations in support of these principles of business conduct is commended.

15. Business should render restrictive legislation unnecessary through conducting itself so as to deserve and inspire public confidence.

IX. The Ethics of Contracting

Many engineers are engaged in contracting either as principals or as assistants to contractors. The ethical principles adopted by the Associated General Contractors of America (31) (A. G. C. A.) are suggested as applicable to the engineer in such relations. In presenting this code, slight changes have been made to make it applicable to all contractors and not only to members of the A. G. C. A.

The working principles by which all contractors should be governed in their relations with client owners and the public, with other agencies of construction, and with members of their own profession, are as follows:

A. Owners and the Public.—Fair and bona fide competition is a fundamental service of the industry to which clients and owners are entitled. Any other method in restriction thereof is a breach of faith toward all contractors and a betrayal of these principles; but the competition cannot serve its legitimate purpose unless it operates under conditions fair alike to owner and to contractor.

Observance of ethical conduct toward the contractor by those who utilize his competitive bidding will be encouraged in proportion as he himself abides by the ethics of fair competition. Only when he respects this code can he reasonably ask others to respect it. Ethical conduct with respect to competitive bidding is defined as follows:

1. Competitive bids preferably should be submitted only when a definite time and place for the opening of all proposals have been fixed, at which all bidders or their representatives are permitted to be present.

2. The contractor's professional knowledge is the result of his training and experience, and if he is called upon for preliminary estimates or appraisals it is proper that he should be paid in the same manner that engineers and architects are paid for similar service.

3. Bidders should neither seek nor accept information concerning a competitor's bid prior to the opening, nor by any method suppress free competition. It is equally improper for the owners to use bids in an effort to induce any contractor to lower his figures.

4. On private work, if all competitive bids are rejected, new bids should not be submitted within sixty days unless warranted by a substantial change either

in the work to be performed, or the market, or other basic conditions affecting cost.

5. The amount of a bid should not be altered after the opening except when substantial change is made in the work, or when further bidding on alternate items is requested. In the event of such change or further bidding, the contractor should bid only on the items specified and should increase or decrease the amount of his bid only in proportion to the change or alternate involved. Any reduction of a bid disproportionate to such change, or the submission of any alternate which in effect produces such a disproportionate reduction, constitutes unfair competition. This shall not be construed as prohibiting the low bidder from decreasing his bid.

6. Contractors should cooperate in advising architects, engineers, and owners with respect to the relative costs of various alternates while plans are being prepared and thus seek to reduce the number of alternates to a nominal maximum.

7. When bids are solicited and received by an owner on a lump-sum basis, no competitor other than the low bidder should solicit the work on a percentage basis, or any other form of cost-plus contract, provided, however, that any competitor shall have the right to accept the work on a percentage basis if tendered him without guaranteed maximum cost or at a guaranteed maximum cost not less than his original bid.

B. Engineering and Architectural Professions.—Local and national cooperation in matters of mutual concern should be the basic policy of all contractors in their relations with the engineering and architectural professions, the purpose of this cooperation being to establish a clear conception of respective functions and responsibilities, to guard against uneconomical or improper practices, and to carry out constructive measures within the industry.

Ethical conduct toward architects and engineers demands the following:

1. Support should be given to all efforts of these professions to maintain and extend high standards of conduct.

2. Contractors should give full credit to the value of the services rendered by the architect and engineer and neither undermine nor disparage their functions or usefulness.

C. Sub-Contractors and Those Who Supply Materials.—The operations of the contractor are made possible through the functioning of those agencies which furnish him with service or products, and in contracting with them he is rightfully obligated by the same principles of honor and fair dealing that he desires should govern the actions toward himself of architects, engineers, and client owners. Ethical conduct with respect to sub-contractor and those who supply materials requires that:

1. Proposals should not be invited from any one who is unqualified to perform the proposed work or to render the proper service, or to whom, in event that his proposal should be the lowest received, the contractor would be unwilling to award the contract.

2. The figures of one competitor shall not be made known to another before the award of the sub-contract; nor should they be used by the contractor to secure a lower proposal from another bidder.

3. The contract should preferably be awarded to the lowest bidder if he is qualified to perform the contract, but if the award is made to another bidder, it should be at the amount of the latter's bid.

4. In no case should the low bidder be led to believe that a lower bid than his has been received.

5. When the contractor has been paid by a client owner for work or material, he should make payment promptly, and in just proportion, to sub-contractors and others.

X. A PERSONAL CODE OF CONDUCT AND ETHICS

The ideals of the individual are so important to his future that every engineer should determine for himself, after mature deliberation, the ideals of conduct and ethics on which he desires to found his future actions. Any code prepared by others can but approximate the ideals even of the writers themselves, and may fall short of covering the different relations and ideals of any individual. It would seem desirable, therefore, for each individual to formulate, in his own mind at least (and preferably in written form), a personal code so that he may review and alter his ideals as his experience seems to require. Such a code was found among the papers of Thomas Van Alstyne, a graduate electrical engineer of Cornell University, after his death which occurred on the job (28). While this code was evidently prepared for his own use, it should be an inspiration to others as well as a lasting memorial to this young man.

> "To respect my country, my profession, and myself. To be honest and fair with my fellow men as I expect them to be with me. To be a loyal citizen of the United States. To speak of it with praise and act always as a trustworthy custodian of its good name. To be a man whose name carries prestige with it wherever it goes.
>
> "To base my expectations of a reward on a solid foundation of service rendered. To be willing to pay the price of success in honest effort. To look upon my work as an opportunity to be seized with joy and to be made the most of, not as a painful drudgery to be reluctantly endured.
>
> "To remember that success lies within my own self and in my own brain, my own ambition and my own courage and determination. To expect difficulties and force my way through them. To turn hard experience into capital for future struggles.
>
> "To believe in my profession heart and soul. To carry an air of optimism in the presence of those I meet. To dispel all temper with cheerfulness, kill doubts with strong conviction, and reduce action with an agreeable personality.
>
> "To make a study of my business. To know my profession in every detail. To mix brains with effort and system in my work. To find time to do every needful thing by not letting time find me doing nothing. To hoard days as a miser does dollars. To make every hour bring me dividends in increased knowledge and healthful recreation. To keep my future unencumbered with debts. To save as well as to earn.
>
> "To cut out expensive amusements until I can afford them. To steer clear of dissipation and guard my health of body and peace of mind as a most precious stock in trade.

"Finally to take a good grip on the joys of life. To play the game like a man. To fight against nothing as hard as my own weakness and endeavor to give it strength. To be a gentleman and a Christian so I may be courteous to man, faithful to friends, and true to God."

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APPENDIX

BIBLIOGRAPHY OF THE LITERATURE ON ETHICS AND HUMAN ENGINEERING

(1) **"The Ethics of the Professions and of Business,"** The Annals of the American Academy of Political and Social Science, May, 1922, Vol. CI, No. 190.

(2) **"Professional and Business Ethics,"** by Carl F. Tasusch, Henry Holt & Co.

(3) **"The Ethics of Business,"** by E. L. Heermance, Harper Bros.

(4) **"Business Ethics,"** by James M. Lee, The Ronald Press Co.

(5) **"The Fundamentals of Business Ethics,"** by N. W. Lord, The Ronald Press Co.

(6) **"Book of Business Standards,"** by J. G. Frederick and Nicholas L. Brown.

(7) **"The Game of Business,"** by J. G. Frederick.

(8) **"Arbitration and Business Ethics,"** by C. F. Birdseye, and Charles L. Bernheimer.

(9) **"Contracts, Specifications, and Engineering Relations,"** by D. W. Mead, Past-President and Hon. M. Am. Soc. C. E., McGraw-Hill Book Co., Inc., New York, N. Y.

(10) **"Data on Ethics,"** by Herbert Spencer.

(11) **"Adventures on the Borderland of Ethics,"** by Richard C. Cabot, M. D.

(12) Publications of the Yale University Press:
   a. **"Trade Morals,"** by Edward D. Page.
   b. **"Morals in Modern Business,"** by Edward D. Page and others.
   c. **"Every Day Ethics"**—Addresses. Edward D. Page and others.
   d. **"Ethics in Service,"** by William Howard Taft.

(13) **"The Crisis in Character,"** by J. F. Adams, *Harper's Magazine*, August, 1933.

(14) **"Business Ethics,"** by Frank C. Sharp and Philip G. Fox, Appleton-Century Co.

(15) **"Cases on the Legal Profession and Its Ethics,"** by G. P. Costigan, Jr., West Publicity Co., St. Paul, Minn.

(16) **"Ethics of the Legal Profession,"** by Judge O. N. Carter, Northwestern University Press, Chicago.

(17) **"How to Win Friends and Influence People,"** by Dale Carnegie, Simon and Shuster, New York.

(18) **"The Rediscovery of Man,"** by Henry C. Link, The Macmillan Co., New York.

(19) **"Elements of Human Engineering,"** by Charles R. Gow, M. Am. Soc. C. E., The Macmillan Co., New York.

(20) **"Foundations for Human Engineering,"** by Charles R. Gow, M. Am. Soc. C. E., The Macmillan Co., New York.

(21) **"Problems in Human Engineering,"** by F. A. Magown, The Macmillan Co., New York.

(22) **"Leadership in the New Age,"** by F. E. Wolf, The Tuttle Publishing Co., Inc., Rutland, Vt.

(23) **"The Return to Religion,"** by Henry C. Link, The Macmillan Co., New York.

(24) **"How to Think Straight,"** by R. H. Thouless, Simon and Shuster, New York.

(25) Taken with some changes and additions from **"The Games of Business,"** by J. C. Frederick.

(26) **Matthew** VII : 12.

(27) Based on the Code of Chemical Engineers.

(28) **Civil Engineering,** January, 1937, p. 5.

(29) **Year Book,** Am. Soc. C. E., March, 1939, p. 6.

(30) Code of Ethics, U. S. Chamber of Commerce.

(31) Manual of The Associated General Contractors of America.

(32) Manuals of Engineering Practice, No. 5, p. 8.

(33) H. R. Doc. No. 7635; see *Civil Engineering*, January, 1940, p. 43.

(34) **"Code of Practice,"** Manual of Engineering Practice No. 1, Am. Soc. C. E., p. 1.

DISCUSSION

Louis E. Ayres,[6] M. Am. Soc. C. E. (by letter).—This paper presents a compilation of the best thoughts from many existing codes, leavened and supplemented by judgment and experience. The author has gone beyond the scope of the title, and, as stated, he "discusses the vital relation of good principles and good conduct to success in life." Much of this material should be helpful to young men in all professions, as it is essential to right living and sound happiness. The author is fundamental and "old-fashioned" in his approach to the problems of life. He still believes in the virtues of courage, character, hard work, personal energy, and will power; and he discounts the validity of some modern theories of psychologists which tend to excuse failure and to discount success on the basis of heredity or environment. One must conclude that the author's thesis is one that will not be accepted widely in an easy-going generation when so many seek success without effort, and as their due; but it is at such a time that the reiteration of fundamentals becomes important.

In reading the paper one is impressed by its scope and detail. In fact, it is easy to be over-impressed by the enumeration and comments on so many essentials to success. How can the ordinary individual hope to measure up to the standards set forth by the author? Is there not a possibility that many a younger man, after conscientious study of this document, may be discouraged by the requirements of a professional career? Obviously, the point to emphasize is that all codes, like the Ten Commandments and the Sermon on the Mount, are the ideal objectives to be sought but not the ends that are easily or immediately achieved. Paradoxical as it may sound, success itself consists in acquiring those very elements in character and personality that lead to success. In other words, the road is open to him who has the will power to persevere in his own self-improvement. The author's paper then becomes a textbook for the student, and a practical guide for any one who would keep in mind the "rules of the game" if, in the end, life is to reward him to his satisfaction.

The author calls attention to the importance of budgeting one's time to avoid the often wasted hours, and suggests that one half of the out-of-office time of the young engineer may well be devoted "to study that is collateral to his job." What kind of study is "collateral" to an engineer's job? Is such study to be confined solely to technical fields? Is it not equally or even more important that the young engineer make an early and systematic start in the reading of good literature, history, and other fields that will broaden his interests. It is also desirable that engineers become active as citizens and that they participate in politics and other group activities in their communities. They are also urged to be active in engineering organizations—local, state, and national. With such a wide range of demands on the out-of-office time of the engineer, the budgeting of his excess energy becomes of prime importance. It is absolutely essential, therefore, that he choose with discretion among the many activities that intrigue his interest, to the end that effort may not be spread too thin, or health injured by over-strenuous living.

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[6] Cons. Engr. (Ayres, Lewis, Norris & May), Ann Arbor, Mich.

The writer is an advocate of the idea expressed by Section III, Paragraph 10, having to do with the "Ethics of the Expert Witness." The engineer is frequently called upon to give testimony in courts or before commissions and in so doing often finds himself under pressure to be a partisan, to take the "side" of his client, and to present figures and express opinions that are biased and either more or less than the truth. He is the victim of a system which depends for results on the presentation of extreme viewpoints, often to the extent of falsity, on the theory that one half of the sum of zero and infinity results in a real integer. The Code of Ethics of the Society states (29) that it shall be considered unprofessional for an engineer "To advertise in self-laudatory language." In the light of this rule, who can but deprecate the extremes to which expert witnesses sometimes permit themselves to be led by an attorney in court in presenting their experience record and their qualifications for the expression of expert opinion? How frequently the balloon of their extravagant claims is punctured by the opposing attorney on cross-examination. Obviously, these statements are not to be interpreted as any general accusation of engineers as expert witnesses. As a rule, it may be asserted, they do conduct themselves in court on a relatively high plane of professional conduct; but the exceptions are sufficiently numerous to cause an opinion to prevail that almost any claim can be supported in court by an expert.

The writer is not prepared to suggest the complete statement of principles that should be incorporated with this paper to cover the proper conduct of expert witnesses. One may hope that Professor Mead will be willing to do so. It has long been the opinion of the writer that there should be an addition to the official Code of Ethics (29) approximately in the following language:

> It shall be considered unprofessional and inconsistent with honorable and dignified bearing for any member of the American Society of Civil Engineers:
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> 8. To give expert testimony in court, or before a commission, differing in substance or implication from what he would present and defend before a technical meeting of this Society.

The writer would close this brief discussion by a word of emphasis on service as a motivating factor in human action if the satisfaction of success is to be fully realized.

The word "service" has a hackneyed flavor. It has been overworked. It suggests to many minds mainly social service work in the slums and among the poor and the unfortunate. In its true sense, however, it comprehends all actions that preserve and develop civilization. It is the essence of the Golden Rule (26). It should motivate man's actions toward the family, the church, and the state. The first objective of the employer to the employee, the employee to the employer, and the engineer to his client should be service. The engineer's interest in technical societies should be based on his desire and willingness to serve, and it goes without saying that this great Society has but one prime objective—namely, to be unselfishly useful to the profession and to the public. The criterion of service should be ever present in all the acts of the Society members, both those who aim at technical improvement and those concerned with the personal welfare of members. What other justification can there be, for example, for registration of engineers other than the ultimate benefit to the public whom the engineer aims to serve?

Considerable thought has been given in the past to the preparation of arguments that may convince young men that they should join the Society. One may point out the value of the publications, of meetings, and of personal contacts with successful engineers. Perhaps such matters as jobs, salaries, and wages may also be presented as inducements; but, if the thought of service is omitted from the arguments, a most important item has been overlooked. Young men, in particular, may be appealed to on this basis; and older men, who have not become hardened and cynical, generally accept this theory of living as the one most conducive to success and real happiness. A proper emphasis on the ideal of service is desirable, therefore, in this statement of "Standards of Professional Relations and Conduct."

IVAN C. CRAWFORD,[7] M. AM. SOC. C. E. (by letter).—In completing and placing before the engineering profession this valuable compilation and interpretation of the ethics which should govern engineers in their relationships, one with another, and with their clients and the public, Professor Mead has accomplished a most worth-while task. Furthermore, he brings the subject vividly and concretely to the engineering student and the young engineer. Particularly pleasing is the emphasis placed upon personal energy and will power as essential elements for success, and the necessity for the student and the young engineer to budget leisure time carefully.

As the author has stated, in general, the ethical principles which he presents are axiomatic. Some educators will disagree, however, with his statement (heading, "I. The Engineering Student and the Young Engineer," paragraph 5) that "On the other hand, he should not strive too much for originality." These persons will argue that engineering students particularly have been, and are, subjected to the type of training that does not encourage originality, and places a premium on the following of textbook material. Such criticisms no doubt have some foundation, but the fact still remains that it is difficult for a student or an inexperienced engineer to be original when his knowledge and experience do not give him the necessary familiarity with his field to warrant a departure from what is considered to be accepted good practice. Professor Mead's discussion of this point seems to be thoroughly sound.

Of especial interest is the section of the paper devoted to "III. Public Relations of the Engineer." Surely every one will agree with the statement (paragraph 6) that "The engineer should discourage, in every legitimate manner, the construction of public works that are economically unsound for the community, state, or nation." However, the determination of economical unsoundness will often be a matter of considerable difficulty, although, as demonstrated within the past few years, the outstanding examples of such unsoundness can usually be shown to the public. Much credit should be given to the members of the Society who have expressed their opinions fearlessly regarding unworthy projects, and who in certain cases have contributed to the abandonment of economically unsound developments.

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[7] Dean, College of Eng., Univ. of Michigan, Ann Arbor, Mich.

This paper, because of its completeness and clarity, as well as the importance of the subject matter, could well be incorporated in engineering administration courses or in other courses that deal with the subject of professional ethics.

Walter H. Wheeler,[8] **M. Am. Soc. C. E.** (by letter).—"Professional Conduct" is a subject of vital importance to the engineering profession. May the day come speedily when the profession will be guided by such high standards as the author has outlined so completely. At present, the profession seems to be far from that goal.

The writer agrees heartily that it is unprofessional to accept engagements for which remuneration is too low, the standard being that which was approved by the Society (32) in September, 1930. He also agrees that it is unprofessional and not in the public interest to compete for engagement on the basis of the price for services. Unfortunately, competition on that basis is far from unknown, particularly in connection with engagement of engineers for public projects.

There is another type of competition which the author did not mention and which the writer believes is both legitimate and stimulating in its effect. This type of competition is provided for to a limited extent by a bill introduced in the Congress of the United States on January 3, 1940, by the Hon. John G. Alexander of Minnesota (33). One section of the bill authorizes any bureau or department of the government to employ engineers in private practice to prepare alternate designs for the larger projects on a semi-contingent basis. Such procedure does not displace or attempt to displace the engineer of the project. It opens the way for enterprising engineers, who know how to compute costs in detail and to design so as to get the lowest construction costs without reducing strength, durability, or utility or affecting appearance, to cooperate with the engineer of the project by submitting alternate designs on a contingent basis. The payment of the engineer's fee is contingent upon showing a substantial saving by the adoption of the alternate design. The saving is determined by contractor's bids. Thus the engineer of the project may render better service to his client by reducing the cost of the construction, and the engineer who submits the successful alternate has the opportunity to realize on his ingenuity. Encouragement is given to enterprise, ingenuity, and progress. It cannot be justly claimed that there is anything undignified or unprofessional in such a procedure. There is the further consideration, as Professor Mead has stated, that engineering is a business as well as a profession.

The author's comments upon educational requirements for engineers are interesting, and particularly his statement (see heading "Introduction: Technical Training and Technical Ability"), "that the study of professional subjects in any great detail is inadvisable, because all such subjects are changing rapidly and few schools can keep in step with those changes." This is worthy of the most careful consideration by educators. The writer believes that preparation for a professional career in engineering should include more general education of a cultural nature. Thorough grounding in the fundamentals of English, public speaking, government, economics, and the law of contracts, as well as in mathematics and the sciences, is desirable. Why not enlist the aid of men engaged in private practice to lecture to students on professional subjects?

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[8] Designing and Cons. Engr., Minneapolis, Minn.

The author has referred briefly to patents. The patent laws of the United States have been one of the most stimulating causes of technical progress. The way of the inventor is not an easy one. Comparatively few inventions ever achieve commercial importance. Some of the greatest engineers have also been inventors, and the results of their efforts have been of tremendous benefit to mankind. Civilization owes much to them, and the profession should take pride in their accomplishments and should encourage them.

The profession is indebted to Professor Mead for his excellent presentation of this subject.

Charles R. Gow,[9] **M. Am. Soc. C. E.** (by letter).—The several concepts, principles, and rules of conduct so well enunciated by the author of this excellent presentation might well be applied indiscriminately in the case of every young man entering upon his life's career, whether it be in the field of professional, business, or public service. In general, they constitute fundamental requisites for success in life and for this reason are entitled to most serious consideration.

A grave peril confronts the people today because of the modern tendency to emphasize the importance of individual rights and privileges to the greater or lesser exclusion of consideration for such reciprocal essentials as duties and obligations. Economic progress and human well-being have made startling advances over the past one hundred and fifty years, largely by virtue of the willingness of the people to accept sacrifice, hardship, and the exercise of patience as the cost of such blessings. In recent times, however, there has developed a tendency to decry the success of those who heretofore have profited materially from strict adherence to old-fashioned principles, and to insist that the fruits of their efforts and sacrifices should be shared generously with their less fortunate contemporaries, including the indolent, the inefficient, and the wastrels of society.

It is undoubtedly true that a professional calling requires a somewhat more rigid code of ethical behavior than might be deemed essential in some other walk of life. The obligation of the engineer toward his client, for example, partakes, no doubt, of a more confidential character than that which exists, say, in the business world between the ordinary seller and purchaser. Men of questionable character have been known to prosper for considerable periods in the business world; but such instances have been extremely rare within the engineering profession since, as a group, its members have been too jealous of its good name to tolerate the presence even of "a single black sheep within its fold."

It is relatively easy to compile and to suggest rules of conduct for the general enhancement of the standing of a profession. Unfortunately, however, nature has endowed mankind with such a variety of diverse traits of character that complete cooperation in support of any proposed code of behavior is difficult to obtain, even among the members of relatively small and select groups. However, it is not altogether impossible in a cooperative society to subject such human frailties to a certain measure of control and restraint through the medium of effective educational measures. At one time or another most engineers have had occasion to subordinate their natural impulses in favor of another course because their intelligence has convinced them that self-discipline and cooperative action will be likely, in the long run, to secure greater rewards. In fact, in the absence of such a realization among intelligent human beings, there could be no civilized society.

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[9] Pres., Warren Brothers Co., Cambridge, Mass.

Therefore, it would appear to be largely a matter of educating the individual to a full appreciation of the necessity and desirability of conforming to uniform standards of behavior which have come to be generally recognized as representing the consensus of opinion among experienced men engaged in a similar calling. This suggests the desirability of inculcating these principles in the minds of embryo members of the engineering profession at the earliest possible period in their careers. It seems somewhat incongruous to delay such efforts until the individual engineer has attained some degree of prominence in his work; yet it is the writer's observation that relatively few become seriously impressed with the importance of this subject until the magnitude of their responsibilities brings them into direct contact with it.

Should it not then become the aim of technical schools and colleges to introduce the subject of professional standards into their curricula as a component element in the students' educational equipment? If such a code of practices and relations as that outlined by the author, or with such modifications or amplifications as might finally be determined upon by the profession in general, could be provided as a basis of a college course in practical ethics, it should be possible to produce student graduates who would be as well grounded in their understanding of conventional professional practice as they now are in technical knowledge.

Several years ago the writer was induced to accept, for a limited term, a newly created chair in "Humanics" at the Massachusetts Institute of Technology, at Cambridge, Mass., and to set up the course of instruction pertaining to the subject. The purpose of the course was to advise the students respecting the nature of the many human problems, which they might expect to encounter in their subsequent relations with other individuals, growing out of their employment experiences. Since it was a pioneer effort in an unknown field, some doubt was entertained as to the degree of interest such a course would elicit from the student body. It soon became apparent that there was an enthusiastic desire among them to acquaint themselves with all phases of the subject. The course has now been in existence more than ten years, and the writer is advised that it continues to be one of the most popular subjects offered.

This experience leads the writer to believe that any subject that is likely to affect the student's future success in his subsequent professional life will make a strong appeal to him during the educational period. There is surely no better time or opportunity to implant fixed principles of conduct in his mind and thus to fit him in advance for an automatic adjustment to the accepted ethical code of his future associates and contemporaries.

It is the writer's opinion that Professor Mead's suggested set of standards constitutes an excellent starting point for the establishment of the recognized principles which make an acceptable basis not alone for the guidance of practicing engineers but quite as well for the initial instruction of the engineering student.

J. T. L. McNew,[10] M. Am. Soc. C. E. (by letter).—Someone has said: "The young have been preached to, lectured to, taught, exhorted, advised. They have seldom been talked to." In this paper the author has done an admirable job of "talking" ethics to young men in a straightforward manner such as to serve as an inspiration to them. The paper was needed urgently by the entire profession to fill a vacant spot in the training of young men. The tenets expounded are all based upon honesty, industry, study, perseverance, self-control, tenacity of purpose, and righteousness—the values of which, in life, are not debatable.

As a teacher of classes of senior engineers in a course in professional relations for a number of years, the writer agrees with Professor Mead as to the need that young men have for information of this kind. The only question the writer would ask is, "Why is not the paper even more inclusive?" and he realizes that great length decreases value.

The writer is thoroughly convinced that an overwhelmingly large majority of engineering students desire earnestly to follow their chosen professions in such a way that they may be better engineers in every sense of the word than their predecessors. They are inherently honest and are anxious to "play the game by the rules," not because of the rules, but because it is right that they so "play." They are much interested in statements of right policy as well as in the viewpoint that makes the policy or ethics right. To insure that they are privileged to obtain such information in the proper way is the obligation of teachers as well as practitioners. In an overcrowded curriculum, however, it has been, and is, an easy matter to neglect stressing the value of righteous professional conduct, although it is always admitted that failure to conform to the recognized ethical practice is the surest way to fail.

To the successful engineer, ethical practice becomes habit and habit comes from training—by doing the same thing correctly over and over again until it appears that the right way is the only way an assignment can be done. No better place exists for developing the habit of good professional conduct than on the campuses of American colleges where four years of "practice living" can be had and where mistakes may be corrected in ways other than the wrecking of an entire life. Correct habit depends upon self-discipline and upon the ability to impose self-denial at times. Literally, one may sow acts and reap habits, while the sowing of habits makes possible the reaping of character. The repeating of proper acts requires thought in the selection of the acts if good habits are to result. The absence of thought results in bad habits because it is easier to do careless acts. The writer, therefore, would add to any young man's code:

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[10] Prof., Civ. Eng. Dept., Agri. and Mech. Coll. of Texas, College Station, Tex.

> "Make your habits; do not let them just 'happen.'"

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Every accepted professional code of practice agrees with the pronouncement that self-laudatory advertisement is dishonorable. With this the writer would include,

> "Self-overvaluation is offensive and will always be resented and over-discounted."

One of the first dilemmas that comes to a young college graduate often involves the ethics of securing employment and of accepting a job after he has had several applications to different firms favorably received. It is not always possible for the student's first job to be in the exact field of endeavor that he has thought he would enter. In most cases, no doubt, he will accept the first employment he may be offered in his general field of endeavor. In accepting this first employment, the student often does so with the mental reservation: "I'll quit this job the minute I find something better to my liking," and all too often he does just that, with never a thought as to the ethics of his action. Perhaps this employer has given the young man a job that was to be of considerable duration and his first months of work constitute a "breaking in" or training period; as such, it is a liability to an employer. It is likely that the employer considered several applicants, some of whom were sincere in there desire to secure the job. Under such conditions the young engineer quits after a short period of service, and the employer is materially damaged and suffers valuable time lost. Of course, the employer takes care to prevent a recurrence of the episode by one of two precautions. He either becomes suspicious of all young college graduates or decides that henceforth he will waste no time on graduates of this or that engineering school. The writer is of the opinion also that such an employer may question the value of the services of some engineering educator, perhaps justly so.

The entire question of proper conduct in seeking, securing, and holding employment by young college graduates is worthy of having a more complete treatment and, although such conduct is a matter that most any graduate should be capable of handling ethically, unfortunately it is one of the first to be violated.

A young college senior wants a job to follow commencement; he writes numerous applications and interviews personnel directors. After a weary time of waiting, during which he receives several replies stating that there are no openings, he discovers an offer of a job in his mail and in high spirits he accepts hurriedly, even if commencement is still six weeks or more away. Now that he is assured of a job, his cares are over until several weeks later when he receives another offer of a job at $25 more per month. Of course a young engineer asks himself the question: "Now, what shall I do?" He needs the best job and "best" to him at the moment may and often is the one that will help him the most in paying his college debts. Shall he accept the best paying job and write a letter to retract his first acceptance, or just what can he do? The old practitioner knows the answer; probably the student also knows, but it is difficult for him to realize that a code of proper conduct may appear to penalize him. As a matter of fact, the faithful adherence to a worth-while code does perhaps "cost" something in a sense, but when the subject is analyzed, one invariably comes to the conclusion that in the end the "price" is cheap indeed and the paying of it gives one his self-respect.

The writer commends the author for including one section devoted to the ethics of business relations. The practice of so-called sales-engineering, wherein the self-styled or trained engineer becomes a salesman in the distribution of construction materials, furnishes a field of endeavor for many men. Is the change of status from engineer to salesman justification for an individual to discard an engineer's code and set up a rule which substantially says: "Success is to be had only as sales increase, regardless of how the increase may be accomplished"? Obviously, honesty can have only one meaning, and misrepresentation in sales practices is precisely as dishonest and unethical as false design, careless work, or other indifferent engineering practice.

Good reputation is something that is cherished by every human; and good reputation results not only from a life well lived, but also from experience gained in work well and properly done. Proper ethics adds to reputation, and long adherence to proper standards of professional conduct increases reputation immeasurably. One may not be the right kind of engineer today, the salesman tomorrow with a different code, and a reputable engineer again next week. Good reputation, therefore, is the result of good habits and requires time, diligence, and patience to establish. Gained otherwise it may not and probably will not endure. Josh Billings stated[11] it correctly, although perhaps inelegantly, with the remark: "Lasting reputashuns are a slow growth; the man who wakes up famus sum morning, iz apt to go to bed sum night and sleep it all off."

W. L. Waters,[12] **M. Am. Soc. C. E.** (by letter).—Professor Mead's comprehensive paper can be divided into two parts: (1) That relating to the technical education and development of the young engineer, which is the usual advice given to college students; and (2) that relating to personal ethics and conduct. Part 2 is somewhat long and detailed so that, perhaps, it loses some of its "punch." It should be sufficient to say that "every engineer should at all times think and act in accord with the highest principles of personal honor." If anything more elaborate is desired, nothing could be better than Thomas Van Alstyne's code of honor which is given at the end of the paper (28). It is truly a classic, and all must feel indebted to Professor Mead for spreading it on the Society records.

Some may consider that in this most excellent paper (which it is stated is principally for the younger engineers) there is a slight touch of the "holier than thou" attitude of the older men. It is youth which must preserve the high ideals and standards of honor. By middle age most men have become somewhat materialistic; and if they are honest with themselves, they look back with some longing to the high ideals of their youth. In thirty-five years of experience the writer has never once known an engineer less than thirty years of age to act professionally in any other way than in accord with the highest standards of honor. Would that he could say the same of those older than forty.

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[11] Josh Billings' Farmers' Allminax.

[12] Cons. Engr. (Bury & Waters), New York, N. Y.

The young engineer is told that he must not compete for work on the basis of "cut-rate" fees. One of the most highly respected consulting engineers abroad "broke into" a certain line of work by cutting his fees below the accepted standard percentage, and was criticized for so doing at the time; but now all that is forgotten and he stands at the top of his profession. Others have done the same. So the temptation for young engineers to do likewise is evident.

CHARLES F. SCOTT,[13] ESQ. (by letter).—From Table 1 in Professor Mead's paper it appears that only 25% of the ideal equipment of the engineer is technical knowledge and skills whereas 75% concerns character and judgment and other personal qualities. Everyone should read the list and study it. The knowledge of fundamentals and much of the technique required in practice are included in the college curriculum; it is what is prescribed and taught to the undergraduate student. However, even a perfect scholastic grade in them does not insure a successful engineering career. The other 75% are the personal qualities, the things which cannot be forced into the student, but which he must develop within himself.

In his thorough analysis Professor Mead presents ten different headings, "Codes" or "rules" with an average of a dozen items under each. In a preliminary paragraph he refers to these items as "the following principles" and his repeated reference to Item (26) in his bibliography declares in effect that the rules are simply applications of the Golden Rule. The student may do well to take to heart the early items specifically addressed to him, and regard the remainder as an example of the difficulty of expressing intangible principles by rules. There is a professional attitude, a professional spirit which is the essence of a true profession. The scores of rules are indications of the principles as applied to particular cases; the student should use them to aid him in understanding the professional spirit and cultivating the professional attitude.

M. J. EVANS,[14] ESQ. (by letter).—Character building and personality development have been taken entirely too much for granted in the engineering profession. The fundamentals involved are admitted by all thinking persons, but the emphasis has been lacking.

The most critical problem facing industry today is a definite shortage in the supply of men capable of filling the bigger positions, particularly those earning more than $10,000 per yr. This shortage is due, not to a lack of ability, but to a lack of understanding of the simple fundamentals of what might be termed, for lack of a better phrase, "Human Engineering." In the final analysis the problem that faces the successful engineering executive is that of dealing with people, and of securing their hearty cooperation in the accomplishment of any objective.

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[13] Prof. Emeritus of Electrical Eng., Yale Univ., New Haven, Conn.

[14] Evans Associates, Inc., Chicago, Ill.

It is generally admitted that at least 75% of a man's success is due to these intangibles, and yet the profession has given them little systematic thought and study. If this had been done, the tremendous economic waste of the five years 1935–1940, that has arisen through strikes and lack of understanding between management and labor would have been eliminated. The engineer has brought mechanical operations to a high degree of efficiency; but he has not carried his share of the load in the matter of increasing the human understanding in the plants of the United States. To quote Dean Dexter S. Kimball of Cornell University, "Our wisdom has not kept pace with our knowledge."

In numerous contacts with young engineering students in American universities during the five years since 1935, the writer has been amazed at the complete ignorance shown by many young men of the principles which the author has so forcefully expressed in his paper. Not that they have never heard of these principles—these vital fundamentals of life have seldom been stressed as an essential part of the engineering profession.

It would seem that one of the most effective ways in which this can be done is through the sharing, on the part of senior engineers, of their broad experience and understanding. Younger men are very anxious in most cases to do the right thing; but they cannot be expected to find the most advantageous road if those who have gone before and who have been successful do not prepare the necessary road maps.

R. L. Sackett,[15] M. Am. Soc. C. E. (by letter).—The statement by Professor Mead on the ethics of engineering is very complete and most admirable. In addition to reviewing the relations of engineers with each other, with clients, and with the public, he has emphasized the importance of personal qualities on the future advancement and usefulness of the engineer. In 1915, when the first investigation of engineering education was made by Professor Mann[4] for the Carnegie Foundation for the Advancement of Teaching, some 5,000 engineers focused attention on the greater importance of character, judgment, accuracy, industry, the ability to work with others and to direct them, as compared with technical knowledge. The "Code of Courtesy and Personal Conduct" formulated by Professor Mead is a very thoughtful and thorough digest of principles which each engineer, young or old, ought to make cardinal virtues.

Much is said today about the importance of social science as a subject that should be included in the education of all engineers because of the influence of engineering works on human welfare and behavior. This code of personal conduct is of more importance as the foundation for good social relations than are most of the textbooks on social science. Human relations begin with the human being and his personal balance, his restraint, poise, good humor, outlook, attitude toward others, his sense of responsibility, his devotion to the profession, and his realization of his debt to society. The technical training of engineering graduates is adequate. Its quality has been attested by a careful survey[16] conducted by the Engineers' Council for Professional Development.

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[15] Dean Emeritus of Eng., The Pennsylvania State Coll., State College, Pa.

[16] Seventh Annual Report, E.C.P.D., October, 1939, p. 6 *et seq.*

The colleges have always been aware of the importance of character, and have attempted to protect and develop it. There is now evidence of an awakening to a sense of responsibility for guidance toward personal attributes and technical competence which together characterize a profession. Combined, they constitute a chapter in social advance which marks the engineering profession as qualified for public recognition. Education should contribute toward personal development and this emphasis by Professor Mead will encourage engineering schools in further efforts to assist young engineers in realizing professional status.

ALONZO J. HAMMOND,[17] PAST-PRESIDENT, AM. SOC. C. E. (by letter).—Both professional relations or principles of ethics and code of conduct or practice are combined in this paper. The code of practice adopted by the Society, January 18, 1927, states (34) in its "Preamble": "Any Code of Ethics is founded on the Golden Rule," so the author in his "Introduction" has developed his theory of the philosophy of life which, for the young man and student, forms an exceedingly valuable part of the paper, and is worth the most thorough study.

Engineers are faced with the query, "Is engineering a profession?" The answer in some instances has been a question mark, based on the assumption that the Society has not acquired, nor do the membership in general subscribe to, an established and agreed "Code of Ethics." Critics state that engineers have not advanced to the stage of the so-called three learned professions—theology, law, and medicine—due to this lack of an established and adopted code.

Every one, no doubt, will subscribe to the need of the brief "Code of Ethics" embodied in the seven paragraphs printed in the Society Year Book (29); but has the Society attempted to instill into the mind of the student and young engineer a philosophy of life that will lead him to follow, naturally, the precepts in that brief code? The great importance of this paper is the presentation of principles applicable to all professional positions and its emphasis to the young man. Of greater importance to the engineering profession, as such, is the opportunity thus afforded to take advantage of the study as basic material; to cull from it a complete code of what may be termed "Principles of Engineering Ethics" or "Canons of Professional Ethics" (whichever heading meets the fancy); and to set up a code or set of rules to be approved by the members which will satisfy the requirements of a professional spirit.

Referring briefly to the paper to indicate what the writer would include in such a code:

As a preamble he would make the following statement:

"As Engineering is the science of controlling the forces, and utilizing the materials of nature for the benefit of man, and the art of organizing and directing human activities in connection therewith, the practice of Engineering is a Profession."

Following such a preamble, the writer would make a part of it the paragraph of the paper entitled "Requirements of the Profession." Then following and becoming part of the code with such mention as may be deemed desirable, the following sections would be included: "III. Public Relations of the Engineer," "IV. The Personal Relations of the Engineer," "V. The Engineer's Relations to Client or Employer," "VI. The Engineer's Relations with His Employees," and "VII. Relations of the Engineer with the Contractor."

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[17] Cons. Engr., Chicago, Ill.

The substance of these five sections, together with the suggested preamble, would form the long-desired and long-needed "Principles of Engineering Ethics" —a complete code for the engineering profession.

In his deeply interesting Romanes lecture, Prof. Thomas H. Huxley has stated the opinion "that the Ethical progress of Society depends upon our combating the cosmic process which we call the Struggle for Existence." Since, as he adds, "we inherit the cosmic nature which is the outcome of millions of years of severe training, it follows that the 'Ethical Nature' may count upon having to reckon with a tenacious and powerful enemy as long as the world lasts."

May the combat at this time be successful in the adoption and application of the "Principles of Engineering Ethics."

A. B. McDaniel,[18] M. Am. Soc. C. E. (by letter).—A notable contribution has been made by Professor Mead to the literature on professional engineering relations. His scholarly statement is worthy of careful study by all practicing engineers and especially by the younger members of the profession.

From observation, both within and without the scholastic field, the writer has been impressed with two outstanding tendencies of engineering education:

1. To cram the student with large doses of technical knowledge; and
2. To stress the importance of material success.

Most engineers whom the writer has questioned have stated that knowledge required in their work has been acquired largely from their experience since graduation. Should not the engineering schools concentrate their resources and efforts on mental training—the development of those qualities referred to in Items Nos. 1 to 4, inclusive, of Table 1 entitled "Characteristics Necessary to Success in Engineering"?

Many engineers go through life without a clear understanding of how to live. This fundamental should be taught to engineering students, beginning with the freshman year, through a comprehensive course in human and engineering relations. Such a course should be given by a practicing engineer with a long and varied experience and the ability to inspire his hearers.

How few people, and especially engineers among professional men, realize the importance of contacts and the maintenance of friendly relations with all sorts and conditions of people! One of the writer's classmates owed his start in life and education to the kindly services he rendered to an elderly man during the latter's illness on a lake boat trip between Buffalo, N. Y., and Detroit, Mich. The sick man, John D. Rockefeller, Sr., was so impressed with the unsolicited and helpful services that subsequently he aided the engineer in becoming one of the leading industrial executives of his time.

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[18] Cons. Engr., Washington, D. C.

Too much emphasis cannot be placed on the engineer's duties and responsibilities as a citizen of the community in which he lives. He should become an active member of his local religious, civic, and professional organizations and should utilize every opportunity to make contacts, participate in meetings, and serve on committees. He should represent his profession with dignity, efficiency, and integrity on all occasions.

Over generations of students, the tradition has been to do just enough work "to get by." Fortunately, this practice is not so prevalent in engineering schools as among other schools of college grade. The writer remembers an acquaintance who boasted that he received his baccalaureate degree from a well-known college without even opening a textbook; he majored in the "cramming" courses of a large and flourishing private school in the vicinity of the campus. The student should be encouraged to do more than the assigned lesson, so that later in life he will have acquired the habit of going beyond his immediate task.

C. B. BURDICK,[19] M. AM. SOC. C. E. (by letter).—The biologists teach that man made the greatest stride in advance when he rose up on his hind legs and thus freed his hands to grasp things. Since that far off day, in common with all organic life, he has exploited the animal and vegetable kingdoms, so far as he could, to satisfy his wants. Also in common with the higher forms of animal life the growing brain very soon developed the value of a deference to its kind. Thus customs were formed; then came the common law, and the statutory law. These things came about because they were deemed of value. Civilization is only possible because of them.

Thus it would seem proper that the engineer, gaining his livelihood in a specialized calling, should have a light to guide him especially applicable to that calling. Like a sign upon the road, it should be useful to others who travel that way. The Society is fortunate that the author has taken the opportunity to formulate a road map of professional relations and conduct. That the subject is important is well shown by Table 1, in which it is the weighted judgment of 5,441 engineers that knowledge and technique are rated at only 25% of the qualities necessary for success in engineering. That the subject is timely is indicated by the probably provable fact that 99% of all published matter relates to knowledge and technique.

The qualities that promote success might all be typified under four heads—character, ability, personality, and knowledge. If equal values are assigned to each, one corroborates the concensus of opinion previously stated that knowledge, although indispensable to success, is overshadowed by qualities subject to cultivation that are given sparse attention in most engineering literature. In this matter it must be conceded that opportunity has an important bearing upon the degree of success. However, inasmuch as success is a goal not subject to exact definition, and must be considered in the light of opportunity, as a rule opportunity can be ignored except by continual striving to make use of such opportunity as may be afforded.

The qualities tending toward success can best be utilized if there are "rules of the game." A lasting success can only be attained if there is a reasonable adherence to such rules. Such adherence promotes the general welfare, and it benefits the individual who practices it. No doubt it is true that virtue is its own reward. There is a satisfaction in it. There is no lasting satisfaction without it. Apart from this fact, however, it pays dividends to the individual to adhere to the rules of conduct. Benjamin Franklin expressed this thought in a homely way when he said—"if the rascals knew the advantages of virtue they would become honest men out of rascality."

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[19] Cons. Engr. (Alvord, Burdick & Howson), Chicago, Ill.

This paper is timely. The views expressed are sound. They are based upon a wealth of experience and thoughtful consideration. In principal they are applicable to most forms of human endeavor; in detail they apply to the specialized calling of engineering in all its branches. The Society would do well to give such matters more attention. In the engineer's quest for knowledge let him not forget to cultivate the qualities that make it useful, and thus promote the best interests of the profession.

JOHN M. HAYES,[20] JUN. AM. SOC. C. E. (by letter).—The sincere thanks of every member of the profession are due the author for his able presentation of such an important topic. Standards of professional relations and conduct should be of vital interest to every engineer—young and old.

The Society has great need for an unofficial code of ethics which touches upon every activity in the engineer's life—one that is personal with which to supplement the present formal Code of Ethics of the Society. This paper should be studied thoroughly by every engineer and every engineering student. It will give faith and guidance to the young and reassurance to the old.

The author's treatment of the subject of opportunity is especially encouraging. Many people seem to believe in an adage that "opportunity knocks but once and returns no more." This is a pessimistic attitude. Some think that "pull" and influence have more to do with success than anything else. It is stimulating to have an authority emphasize the importance of sound character and hard work as factors influencing success. The quicker one learns to expect advancement only after he has earned it by hard work the happier he will be, both in his professional and private life. The advice to make effective use of spare time should be heeded by every young engineer, as should also the advice, "The young engineer should not be a 'yes' man."

Much is heard now of obtaining more public respect for the engineer and the profession. The profession will gather respect when each individual member of the profession lives so as to command this respect. There could be no better way for an individual to command respect than to live by the principles stated in the paper. In order for one to practice these principles in his daily life, he must be constantly on the alert, because it is easy to forget good intentions. The author has previously written:[21] "The persistent exercise of the higher ideals, as a basis of life and action—which is often of even greater importance than a supreme sacrifice to the continued development of civilization—requires continued and unremittent effort which human nature, in the main, is apparently too weak to sustain." Here he puts emphasis upon "unremittent effort."

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[20] Asst. Structural Engr., TVA, Highway and Railroad Div., Chattanooga, Tenn.

[21] "The Engineer and His Code," by Daniel W. Mead, Past-President and Hon. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 101 (1936), p. 1483.

It should be the duty of every member of the Society to read and reread the paper from time to time. Thus, he will keep these principles, which should govern his actions, constantly in mind and eventually their practice will become a habit with him. One must realize that it is not often possible to live up to his ideals completely; but one should never lose sight of his goal. Every one makes mistakes, but if he keeps striving he will finally improve himself. The faithful and conscientious practice of these standards of professional relations and conduct in one's daily life will not only make one a better engineer but a happier man.

G. W. HOWARD,[22] JUN. AM. SOC. C. E. (by letter).—Since this is an age of transition and instability, it is well worth while for every man to pause and consider what principles he lives by. At a time when the beliefs and practices of centuries are being questioned, and in many instances discarded entirely, it is incumbent upon the professional man to examine the status of both himself and his profession in order to preserve his equilibrium. If the title "engineer" is to continue in its rightful place in the professional world, it is necessary for engineers to be governed by the precepts enunciated in this paper. It is impossible for the engineer to bury his mistakes with the ease of the doctor; nor can his errors be rectified by a later decision of a jury or judge, as in the case of the lawyer. On the contrary, the engineer must assume sole responsibility for the success or failure of an undertaking. It is doubly important, therefore, that he understand the situation on adopting the profession.

In general, it seems that confusion may arise either from the lack of condensation of the paper, or from the failure of the author to include an adequate summary at the conclusion. Not only ideas, but at times the sentences themselves are repeated almost verbatim. The repetition of identical material may leave the reader with an indefinite conception. The presentation would be clearer and just as adequate if one general set of codified principles, applicable to all phases of the profession, were used, with the addition of specific codes embodying features pertaining only to one particular phase.

However one may regard the organization of the paper, it is undoubtedly true that the ideas contained therein are indeed worthy. As stated by the author, the ethical principles presented in the various codes are axiomatic. The qualities that make for success in any profession are essentially the same. The "yardstick" of attainment has always been one's native intelligence, character, knowledge, and capacity for hard work, each dependent upon the other. The innate ability of a man is wasted when it is not joined to integrity and industry; and the inverse is true.

On the other hand, correct evaluation of the qualities of success by fitting each into the narrow category of the percentage basis, it seems, is impossible. The individual differences existing between men are too great for the formation of any set system. Success is intangible and may not be analyzed as may a mineral deposit—so much gold, so much silver, copper, or gangue mineral. If it were possible, however, to apply a table of characteristics to an individual, there would still be cause for questioning of Table 1 entitled "Characteristics Necessary to Success in Engineering." The author states ("Introduction: Ability to Apply Principles to Practice"): "The important matter is the great weight given to characteristics on which dependability and integrity rest as compared with those factors common to educational requirements; and yet those latter factors are absolutely essential to the success of the engineer." The table in question, in its milder form, gives 75% of the credit of success to "characteristics on which dependability and integrity rest," and only 25% to those of technical knowledge. This proportion is believed to be in error. When a man is ill, he may be concerned about his doctor's character, it is true; but how much more concerned is the same man about his doctor's technical skill and knowledge! Likewise, when an important project is under construction, it is hoped that the engineer in charge has the proper "perspective of life," but it is demanded that he have thorough technical qualifications. Moreover, if the percentages of Table 1 are to be accepted, the futility of the present engineering educational program must be admitted, and the engineering student should substitute courses in "How to Win Friends and Influence People" for those involving technique.

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[22] Capt., Corps of Engrs., U. S. Army, Fort Belvoir, Va.

The importance of hard work and steady application is emphasized by the author. Since the Garden of Eden, the fallacy has been prevalent that work is a curse. As a matter of fact, the greatest blessing ever to reach mankind is work, it seems. Even for the man of average ability, no art or science is believed too difficult for action to circumscribe. In the words of the ancient proverb: "Work conquers all things." Those great men of the past who have accomplished most had one quality in common—the capacity for hard work. Otherwise, Michelangelo would never have covered the walls of the Sistine Chapel; nor would Galileo have developed the Copernican theory of the solar system; nor would Newton have discovered the differential calculus.

Yet as Stevenson[23] says: "Idleness so-called, which does not consist of doing nothing, but in doing a great deal not recognized in the dogmatic formularies of the ruling class, has as good a right to state its position as industry itself." Leisure time, and its proper use, are in a sense as important as work itself. If a man is to have a well-integrated personality, he should have hours free from his profession in which to develop his individual preference for books, painting, music, or whatever his interest. It seems that as a man grows older and his physical powers decline, he should find added enjoyment and content through the proper use of his mental faculties, as he has learned to use them in his youth. This matter has been covered well by Robins Fleming.[24]

Another characteristic of the successful engineer, which was mentioned but not emphasized, is that of self-reliance. The contemporary psychological theories of self-expression are not subscribed to, but the writer agrees with Emerson,[25] who has said: "The highest merit we ascribe to Moses, Plato, and

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[23] "An Apology for Idlers," by Robert Louis Stevenson.

[24] *Civil Engineering*, February, 1940, p. 77.

[25] "Self-Reliance," by Ralph Waldo Emerson.

Milton is, that they set at naught books and traditions, and spoke not what men but what they thought * * *. Whoso would be a man must be a nonconformist * * *. It is as easy for the strong man to be strong, as it is for the weak to be weak." However much one likes or dislikes himself, he must make the most of what he has, and stand upon his own feet.

The implication is not intended that the proper sense of humility should be lacking. Every man who will trouble himself to view the light of Pythagoras, Socrates, Confucius, da Vinci, and others, cannot help losing his smugness and inflated ego. Furthermore, it seems that one of the greatest advantages it is possible to enjoy is the opportunity of working under men of wide experience. Here one may learn from the actual practical experience of others and thus avoid many of the pitfalls of theory or the trial-and-error method.

C. Frank Allen,[26] **Hon. M. Am. Soc. C. E.** (by letter).—Possibly the first item in any ethics provision is that the engineer should have the proper qualifications; that is, qualifications for success; and the writer was glad to find that Professor Mead had given the formula derived from Professor Mann's investigation of the elements of success.[4] Some years ago, while visiting in Zurich, Switzerland, the writer learned of an address that was made by a professor of a great school of engineering in Paris, France, in which five qualifications for success were presented. These qualifications had a little different slant from those of Professor Mann and were character, intellect, scientific foundation, knowledge of man, and knowledge of engineering. It is useless to discuss the question as to whether character stands at the head. It is there.

To how many, at first thought, does character mean just honesty? But character is much more than simple honesty. To the writer courage stands out; and the best synonym that he has been able to find for character is dependability, which is emphasized by Professor Mead. Taken together, character and dependability provide a picture far better than either, alone. To character the writer would assign courage, steadfastness, persistence, vigor, and adaptability. Even honesty of purpose lacks something if it is not combined with an acute sense of right and wrong. It is not necessary to enlarge on intellect.

The creation of a scientific foundation is clearly the function of the college of engineering. It is not necessary that an engineer should get his scientific foundation through the college; but he must get it in some way. It is somewhat uncertain how readily one can accede to the precedence followed between knowledge of men and knowledge of engineering. Regarding this subject someone has said, "Knowledge is power." That saying has not satisfied the writer as an engineer. It is the use of knowledge that is power. Coal is not power; but when it is used it generates power. Water in a still pond or reservoir is not power; but put into action it yields power. So it is the use of knowledge that is power; but when it comes to the use of knowledge, what can one do with it except in connection with men?

In the Society for the Promotion of Engineering Education some years ago there was a paper presented, "Why Not Teach About Men?"[27] and it produced a profound impression upon the writer's mind. Why not study men? Certain rules can be laid down, among them that one can better understand a man if he is observed when he is off his guard. The important point which the writer wishes to make is that if the young engineer will develop the habit of studying men, he will find that his ability in this direction will grow apace.

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[26] Prof. Emeritus, Mass. Inst. of Tech., West Roxbury, Mass.

Useful knowledge should be based on scientific foundation. It should be sound, and not be fallacy. Then it should be exercised intelligently, and beyond that, it should be exercised righteously, with character.

In practice the engineer must depend very largely upon common sense and judgment; but from the writer's point of view common sense and judgment should be based upon the five qualifications stated. Common sense and judgment are not crude affairs but rest largely on experience and precedent—experience acquired and precedents accepted, both of which should have a scientific foundation and not be lacking in the other four qualifications for success.

The writer would like to see rules of conduct formulated for teachers of engineering in their relations to students. There are relations of engineers to contractors, and relations to others, but the relation of teachers to their students very well merits attention.

The young engineer who is placed in charge of a construction job should enter upon it with the presumption that the contractor he is dealing with is fair and honest. In the long run, with the majority of contractors, he will not be disappointed. If, in a few cases, he finds himself mistaken, here again knowledge of men may well come into play. Then as the work progresses, it is the function of the engineer to cooperate with the contractor by staking out work conveniently, by arranging the necessary sequence of operations, and in other ways by giving opportunity for the contractor to do his work successfully. This should operate to the advantage of the engineer's employer or client. Cooperation on one side invites return cooperation from the other side. The writer likes to see the contractors make money but never at the sacrifice of the quality of the work. He was raised as a strict constructionist of the specifications. In the matter of small favors from the contractor, he personally prefers that it should not be altogether one sided. It suited him for the contractor to smoke about as many of his cigars as he smoked of the contractor's.

As stated in the paper, there is a popular saying to the effect that opportunity knocks but once. The writer does not believe in that at all. It knocks often and one needs to hear the knock every time. Furthermore, there is something of value in the proposition that can be extended. The successful man is the one who takes advantage of his opportunities rather than the man of greater talent who lacks that quality. The writer feels confident that he can trace success due to this quality in two conspicuous cases: One of a man of not outstanding ability who became President of the United States; and another of a man of exceptional ability who became President and an Honorary Member of the Society. Along the same line, the writer advises young engineers not to tie up with the man that is constantly unlucky. Perhaps it is not luck at all. It has been said that a teacher should have a sense of humor and be an optimist. This is not restricted to teachers. In the line of optimism, it is good psychology both to build on, and to appeal to, the best in men.

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[27] "Why Not Teach About Men, the Most Important and Difficult Tools an Engineer Uses?" by John F. Hayford, *Proceedings* of the Society for the Promotion of Engineering Education, Vol. XIV, 1906, p. 198.

There is a chance to improve the understanding of engineers in legal matters. The writer is profoundly impressed with the fact that a lack of knowledge of elementary principles of law on the part of the engineer may be a serious handicap in many cases; and he feels certain that an engineer should know enough of the rules of evidence so that he can collect evidence at the only time when opportunity offers and know what kind of evidence will stand up in court. The engineer should also know much about the laws governing contracts.

In closing, the writer wishes to express his appreciation of the suggestions, examples, and illustrations that are made, which engineers should take to heart. Engineers should become so saturated with the spirit of this paper that it will go a long way to building up a character which will enable them to do the right thing under the various circumstances that occur, and are not here enumerated.

ARTHUR W. CONSOER,[28] M. AM. SOC. C. E. (by letter).—Engineers in private practice will find Professor Mead's paper on professional conduct most interesting and informative. He has developed his subject so well that it is probably presumptuous for a younger engineer to take any exception to what he has written. However, it is the writer's opinion that, under present conditions, very few consulting and practicing engineers in the municipal field can be guided by his recommendation that no solicitation of professional engagements be undertaken (see code "IV. The Personal Relations of the Engineer," item, 19). Unfortunately, solicitation in general practice is rather widespread, and most engineers in general practice probably find it absolutely necessary, if they are to have practice sufficient in amount to enable them to maintain adequate staffs.

Many phases of such solicitation are discreditable in the extreme, and every effort should be made by engineers in general practice to discourage criticism of competitors, the use of shady influence, and other ignoble sale-agent tactics. Certainly bidding for engineering work should be discouraged, particularly in cities where it is evident that low price is to be a considerable factor in the selection of engineers. It seems to the writer only practical that for some years to come solicitation of professional engagements by engineers in general practice must be condoned, but that until such time as that practice can be abandoned every effort should be made to remove from methods of solicitation all unsavory aspects. There is room in the Society for a division of engineers in general practice within which round-table discussion could probably be used to minimize objectionable features of solicitation; and gradually the situation could be improved so that ultimately all engineers in general practice could operate without resorting to active solicitation of new engagements. However, it will always be difficult for competent engineers to enter the field of general practice if they are to be estopped from active solicitation of clients, and perhaps some middle ground must be developed for the benefit of younger engineers who are desirous of entering private practice.

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[28] Pres. and Gen. Mgr., Consoer, Townsend & Quinlan, Chicago, Ill.

George C. Ernst,[29] Assoc. M. Am. Soc. C. E. (by letter).—It is inspiring to read this paper outlining the basic principles for a successful life. The encouragement of patience, diligence, and dependability, combined with independent and critical thought, should promote a steadying influence in these somewhat trying times. The faith that is still held in these qualities by the leaders in the profession will provide the backbone for the future development of the profession of engineering. It scarcely seems necessary to point out that every engineering student beyond the sophomore year should be required to study and analyze this paper.

Those connected with academic matters will find it difficult to read this contribution without becoming aware of the obligation and responsibility which rest upon the engineering faculties and schools in the proper training of students in engineering. Meticulousness in the details of pedagogy and attempts to teach excessive amounts of subject matter during the formative years of the 4-yr curriculum shrink in importance when the need is seen for properly preparing each student for the growth of the qualities described in the paper. Without the belief in such qualities implanted in the younger members, the profession cannot look forward with much hope and enthusiasm.

The engineering schools not only stand practically alone in their opportunity to supply the profession with men ingrained with the proper ideals, but should recognize it as their duty and privilege. The writer believes this type of training to be of unusual importance in the utilization of engineers in public service as well as in private practice and in industry.

The mere presentation of fundamental subject matter on the part of the teacher and its assimilation by the student may be accomplished by the routine methods now advocated and used by teachers of education in the handling of large groups. It should be obvious that such methods cannot adequately initiate the development of the proper ideals in the student to the extent known to be desirable in engineering. The writer is firmly convinced that there is a strong tendency on the part of many institutions, inadvertently, to permit or promote the growth of the student body at the expense of training in those very qualities which are esteemed by the profession. The necessity for having well-organized, adequately manned, and fully equipped classes and laboratories should be evident if engineering teachers are to develop the requisite ideals (as well as training) for an engineering career.

It is evident that this need should not be construed to mean a highly specialized curriculum or a decrease in the dissemination of "scientific-technological" knowledge for the benefit of the so-called "engineering fraternity."[30] The profession of engineering, however, is entitled to a better initial selection of entering students than the present educational mortality of more than 50% indicates. The crowded conditions and the heavy mortality occurring in the first two years take a heavy toll of student morale, and the junior and senior students are adversely affected to a greater extent than many are willing to admit. The present graduates would be far better material if they were to start as a more compact group in the first year without the company of those leaving before the junior year.

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[29] Riverdale, Md.

[30] "Report of Committee on Aims and Scope of Engineering Curricula," *The Journal of Engineering Education*, March, 1940, p. 555.

Frank S. Bailey,[31] Assoc. M. Am. Soc. C. E. (by letter).—In the "Synopsis" preceding the author's admirable paper, it is stated that "The paper has been written more particularly for the younger men of the profession and with the hope that it may supply them with some information, usually not available in college, as to what their action and conduct should be in their professional relations in practical life."

Almost all young men who have been taught elementary principles of good conduct, and who are under the necessity of earning their own living, and possibly of assisting others, enter their first employment with the determination to do their work well and to win advancement. Some succeed brilliantly, others only partly, and some to a very minor degree.

In looking back on their youth and middle age, many older men who have not reached the goals for which they aimed in youth may recall various periods in their lives when they might have used their working and spare time to better advantage. A careful reading and weighing of the precepts for success which the author has presented should benefit any young man who desires and intends to succeed. The writer also considers this paper a most useful one for older men who may be inclined to "rest on their laurels," instead of continuing to give their best efforts to whatever work they may be fortunate enough to obtain.

Some men are naturally born leaders and cannot find any comfort in a subordinate position, but the majority of men, after they have reached or passed middle age without demonstrating exceptional ability, become more or less reconciled to the idea that they do not possess the talents which enable men to rise to the higher positions.

Some boys have an unfortunate start in life. They may be given sound advice by some people and very poor advice by others at a time when they lack sufficient knowledge to distinguish between the two kinds. As they grow in experience they realize that the responsibility for any foolish decisions they make rests largely on their own shoulders, and if they are reasonable they learn to exercise great care in avoiding useless risks of all kinds. By the time they approach the end of their careers they probably acquire sound principles of living. Of course, the earlier such principles are acquired the better it is, but "better late than never."

It is the writer's opinion that in the course of their lifetimes men are likely to reach the positions to which they are entitled, although there may be exceptions to this rule. It seems evident, nevertheless, that men who are naturally gifted as leaders are as subject to heavy blows from fate as those not so well endowed, and they may have to summon more courage and fortitude to survive some of their difficult experiences than those whose lot in life is more commonplace.

The author's paragraph on "Intelligence and Native Ability" recalled to the writer that, while reading the *Journal* of Ralph Waldo Emerson a few years ago, he was struck by the thought that Emerson, at the age of nineteen or twenty, possessed more practical acumen than more normal men attain at twice that age.

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[31] With Stone & Webster Eng. Corp., Boston, Mass.

The writer feels that Professor Mead has rendered a noteworthy service in presenting this paper.

S. A. McCosh,[32] Assoc. M. Am. Soc. C. E. (by letter).—This paper places in the possession of the engineer something of inestimable value. The author, in his "Synopsis," states that the application of the principles set forth will result in the greatest personal satisfaction and success. The writer would magnify that statement to say that their practical application will insure, to a large degree, the accomplishment of the participant's desire or goal of life itself—that is, happiness and contentment, the universal desire of human beings, regardless of how they may express such desire or conceive the fulfilment of it. The worth of such application lies in the fact that it cannot be done on a purely selfish basis. Practise of principles, like the invention or development of something worth while, has an unlimited and unending possibility of human benefit.

There is only one exception to the truism that "nothing is constant except change," and that is the permanence of principles. An engineer's productive worth may end with his retirement or death; but his influence, of which he can never possibly know the full value, will continue long after he has departed. The often unconscious acts, mannerisms, etc., of the older, experienced engineer influence those who see and contact him, to an extent that he seldom realizes or comprehends. The younger engineer, to a lesser degree, is exerting the same general influence.

The principles of justice, fair play, honor, confidence, loyalty, and self-control, applied with common sense, will result in greater happiness for, and more efficient work by, engineers. To make principles practical requires more thought, study, and time than the most complicated technical problem. Witness the slow reduction in traffic fatalities in spite of the efforts of many organizations to make the United States safety-conscious.

John H. Meursinge,[33] Assoc. M. Am. Soc. C. E. (by letter).—Since time immemorial the inhabitants of this planet have been forced to keep up a great number of ever-changing rules, ordinances, laws, and codes in order to make an ever-changing society function properly. Inasmuch as the engineering profession is taking a place of increasing importance in present-day society, its need for a code of ethics is obvious. The elaborate paper by Professor Mead should prepare the way for the adoption of such a code. The standing of the profession, the success of the aims of its organizations, and the engineers' usefulness to society as a whole depend entirely on honest dealing with one's fellowmen. History, unfortunately, has proved that times of stress, such as the present, tend to lower (or often destroy altogether) professional morals. The publication of Professor Mead's paper, therefore, can be called timely.

The lowering of morals is a result of the lack of social security; and as, presently, the younger engineers are subject to more "layoffs" than the more mature, experienced engineers, Professor Mead fortunately wrote his paper particularly for the younger members of the profession. However, he fails to mention the fact that steady jobs will probably do much more for the ethics of these engineers than a well-written code. Although a great part of unethical dealings, undoubtedly, may be attributed to greed, another not so often mentioned part is caused by the fear of poverty and losing one's job. Hence, there is a possibility that a code of ethics published for a profession which has not reached the goal of social security yet may become as weak a structure as a building without a foundation.

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[32] Asst. Prof., Civ. Eng., Colorado School of Mines, Golden, Colo.

[33] Huntington Park, Calif.

The great majority of engineers are employees in the service of a public administration or of private companies. The type of work that many engineers are engaged in makes the permanence of employment rather doubtful. Excuses for cessation of employment can always be found in spite of any corrective regulations that may exist—especially since no powerful societies of engineers take an interest in social welfare. The engineers are like an army scattered over too large a territory. They are poorly entrenched and at the same time are on the firing line; there exists no general staff which directs the defense of the individual soldiers who are subject to attack. Therefore, an engineer who likes to do some clean fighting in regard to some ethical problem which has arisen in connection with his work may feel that the commotion brought forth by such an action may eventually cause the loss of his job. Indeed, only very strong characters under such conditions will value professional standards higher than the welfare of a happy family. Outside of the office or the project there has existed for the last few years a kind of a no man's land in which, during the greater part of the time, hundreds of unemployed engineers compete for the few available jobs in the same fashion as so many hungry dogs quarrel for a few chewed off bones.

Maximum practical results of a code of ethics can be expected only in a well-organized society in which the engineer with a family depending upon him does not have to face the privations connected with unemployment. Hence, the engineer, who is a practical man, in order to improve the morals of his profession should join the millions of Americans who today are so earnestly striving to obtain a social order which provides a living space for all. To make his influence felt the engineer must be able to talk intelligently and in an unbiased spirit about social security and economic problems of importance. His reading and education should not be limited to technical subjects. Engineers should establish themselves as a progressive force respected by public opinion not only for their technical accomplishments, but also for their constructive interests in social and economic problems. Then the weaker brothers of the profession will be members of engineering societies which protect their jobs, their salaries, and their standing; and surely they will not be tempted so much to stray off the straight road of professional conduct.

Once the engineering societies have done their share in establishing social security for their membership, another problem related to ethics should be dealt with. Too many unethical dealings are, and have been, nothing but short cuts to fame and wealth. Success has generally been defined too poorly; and too often it has been expressed in the size of the engineers' salaries or in the amount of publicity some engineers happened to attract. Better norms to measure the dimensions of successful living will have to be found. Professor Mead presents a short essay on the subject written by the late Thomas Van

Alstyne (28), which shows that the meaning of the term success has begun to broaden out beyond material gains. As the definition cannot be expressed in terms of the engineers' cherished mathematics, no two engineers probably will ever agree upon the correct expression. Nevertheless, more and better definitions, published from time to time in the engineering magazines, may improve professional conduct. Of all definitions published thus far, the writer likes the following best:[34]

> "It [success] will be judged by the immaterial and more consumable goods we have learned to enjoy, and by our biological fulfilment as lovers, mates, parents, and by our personal fulfilment as thinking, feeling men and women. Distinction and individuality will reside in the personality where it belongs, not in the size of the house we live in, in the expense of our trappings, or the amount of labor we can arbitrarily command. Handsome bodies, fine minds, plain living, high thinking, keen perceptions, sensitive emotional responses, and a group life keyed to make these things possible and to enhance them; these are some of the objectives of a normalized standard."

Summarizing, it should be stated that the engineers who believe in a code (which is something more than a list of rules fit for framing and hanging on the wall) should fight for:

1. A social order in which competent engineers will find continual employment;
2. The adoption of a code of ethics as outlined by Professor Mead; and
3. A profession which measures the success of life not by the tangible values taken from society, but by the moral values given to society.

Karl W. Lemcke,[35] Assoc. M. Am. Soc. C. E. (by letter).—As stated by the author, "It is self-evident that all men desire success in their undertakings." Some, due to the possession of certain characteristics, are predestined to success, while others are not so fortunate. Just as the student in search of engineering knowledge can save himself much time by studying the knowledge accumulated by others and stored in textbooks, so too the engineer in search of help in building a career can learn from the experiences of those who have gone before.

In the "Synopsis" the author states that "The paper has been written more particularly for the younger men of the profession," and this the writer believes is rightly so. Throughout the early part of the paper, in particular, the young engineer will find considerable information of value, and the latter part will come to be more fully appreciated as he advances in years.

In this connection the subject of advice to the young engineer in the matter of a career, and certain suggestions as to the conduct of life, come so close to the author's subject matter that, were the title of the paper slightly different, the writer would like to comment on them. The writer wishes particularly to thank the author for his "A Code of Courtesy and Personal Conduct."

Under "Public Relations of the Engineer" and "The Personal Relations of the Engineer," "To do unto others as he would that others should do unto him" is a principle which, after testing for more than ten years in the selling of engineering materials for construction, the writer can heartily endorse. It pays far greater dividends than sharp practice, dishonesty, etc.

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[34] "Technics and Civilization," by Lewis Mumford, Harcourt Brace and Company Inc., New York, N. Y., 1934.

[35] With Waddell & Hardesty, New York, N. Y.

Referring to item 19, under "The Personal Relations of the Engineer," it would seem to the writer that, in this age of keen competition, under certain conditions it might not be unethical to solicit work actively, if done in a dignified manner. However, he admits that it would be more satisfactory to have the work come to one on the basis of his reputation and experience.

It seems to the writer that the author presents the case very succinctly when he states in item 12, under "The Engineer's Relation to Client or Employer," "A reputation for high ideals, good personality, honesty, dependability, intelligence, initiative, and professional ability is the foundation of professional preferment." This sentence might well be committed to memory by the young engineer.

Some time ago there appeared an article in one of the technical publications decrying those clauses in specifications which made the engineer arbiter or umpire between his client and the various contractors on a project, this on the ground that the engineer, being employed by his client, was not a disinterested party. Apparently engineers are living up to item 2 of the author's section VII (Relations of the Engineer with the Contractor), as there has been no change in this procedure, and all parties concerned seem satisfied to continue under the arrangement. This reflects great credit on the profession.

On occasion the writer has seen phrases in specifications in violation of item 9 of this section, but these could have been obviated by exercising sufficient care in writing the specifications and drawing up the plans.

While selling engineering materials, the writer has seen many of the items of section IX (The Ethics of Contracting) grossly violated, particularly section *C*, items 2, 3, and 4. The author's code, if lived up to, would certainly place all contracting and subcontracting on a far higher moral and ethical plane.

The writer believes that every one could profit from the remarks in section X (A Personal Code of Conduct and Ethics). The code cited shows the young man to have been one of unusually high ideals. In the present state of things such young men are greatly needed in this world.

In conclusion, the writer feels that the engineering profession owes the author a great debt of gratitude for the writing of this paper. Just how much it will be worth to any individual will depend upon how much time he gives to the study of it and to the endeavor to practise its principles. One thing is certain, and that is that, if every member of the profession were to put its principles into use in his daily life, this world would be a far better place to live in, and the engineering profession would enjoy much more prestige than it does at the present time.

John Sanford Peck,[36] M. Am. Soc. C. E. (by letter).—The writer has always been opposed to fixed, rigid codes of any description, be they religious, moral, or professional. Excessive formalizing can lead only to stagnancy and sterility. In a too strict observance of the letter of the code, the spirit and heart are lost. Witness the fate of the Children of Israel in general and that of the Scribes and Pharisees in particular. Even Herbert Spencer, whom the author cites (10), held that "The sense of duty must diminish as moralization increases."

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[36] Asst. Prof., Civ. Eng., Coll. of the City of New York, New York, N. Y.

The writer places much more reliance in the modern philosopher's definition of "moral" as a guide to conduct. That action is "moral" which is based on a consideration of all its possible outcomes, and that outcome chosen which seems to be the best for all concerned. Every situation is unique. It never occurred in exactly that form before, and it never will occur exactly so again. Certain elements may be identical, but all elements are not identical, and hence no rigid code that gives all the answers can be constructed. Even the Ten Commandments ~~are~~ interpreted to fit practical conditions and are not applied literally. A prescribed code of ethics tends to dull the edge of the moral responsibility of the individual for the results of his actions, and transfers that responsibility to the code itself.

The author, while no doubt stating "good old-fashioned doctrine" (especially in that section devoted to advice to the young engineer), has failed to take cognizance of the fact that this is a changing world. What is regarded as the truth today may become the superstition of tomorrow. The social situation is changing by leaps and bounds. The college graduate of today demands something that will work, and he knows that most of the old maxims have failed badly "to deliver in the pinch."

What profits it, if he have all the attributes set forth by the author and the only job he can get is changing nickels in a subway booth? He has discarded the old cult of the "go-getter," the "success" mania. The profession makes a grave mistake when it publishes a code that carries the implication that every engineering graduate is a potential leader of men. Napoleon said that every private in his army carried a field marshal's baton in his knapsack, but how many of them ever rose to wave it with authority over an army? History is silent on that point.

Instead, engineers should stress the satisfactions that may be achieved in life in the rôle of draftsman, transitman, or assistant engineer. The world has much more need of efficient and well-trained followers than it has of an embittered and disillusioned host of trained "leaders" who failed to achieve positions for which they were unqualified.

To turn to a few more specific comments, the writer is in total agreement with the author on the subject of engineering education (see heading "Introduction: Technical Training and Technical Ability"). The writer has labored for a broad foundation of fundamentals plus humanities in the curriculum, and has fought the trend toward too high specialization.

However, the writer must take issue with the author on the subject of the findings of psychology, especially with reference to individual differences. They cannot be thus lightly dismissed. Glands, the intelligence quotient (I.Q.), and inhibitions play a larger part in the life of the individual than the author will admit. Granted, a strong drive may overcome somewhat the handicaps of a low I.Q., but, on the other hand, a strong drive coupled with obvious inabilities may cause that individual untold misery and disappointment. At the writer's institution, a psychologist sits with the Committee on Course and Standing and has been very successful in diverting those students, who in spite of all personal incapacities want to be engineers, to other fields where they will be more useful.

Finally, the writer must take strong issue with the author when he cites the Golden Rule (26) and states that this is the basis of all sound ethical action. To presume to criticize this ancient law is taken by most laymen as evidence of atheism, radicalism and all other terrible 'isms; but there are sincere and earnest thinkers who doubt its reliability as an infallible guide. In a world where every personality was exactly the same, with the same likes and dislikes, the Golden Rule would be workable, but with the recognition of individual differences its infallibility disappears. Why force ice cream on a man who detests ice cream simply because you would like to be given ice cream yourself!

In conclusion, lest the writer be accused of being merely "a scoffer in the market place" and a low fellow who only drags down but does not build up, he would like to offer in place of the author's first part the moral of the Parable of the Ten Talents,[37] which is, in essence, the doctrine of individual differences. Each individual is responsible only for achievements commensurable with his abilities, but it is immoral to neglect even the smallest degree of ability. In place of the author's complicated code of professional ethics, he would offer this quotation from a well-known modern philosopher: "Ethics, in my opinion, is not a matter of detached rules, but of a few simple principles. These are honesty in thought and action, courage, justice and the loving kindness preached by Christ, but often forgotten by Christians."

Finally, another philosopher, William Herd Kilpatrick, has said that it is better to raise a good question than to answer it. Why are codes necessary?

E. D. AYRES,[38] ESQ. (by letter).—Standards of professional relations and conduct are like international laws—they derive their force from the support of those to whom they apply, and they command respect by reason of their acceptance and particularly by reason of their observance by those whose standing among others commands attention. On the other hand, periods of stress force breakdowns in both professional codes and international law, and in each case the offending individual or nation seeks to rationalize his action.

The code presented is unusually complete and its content is comprised largely of almost axiomatic truths, just as much international law is based upon common sense and stands for justice to both big and little nations. Nevertheless, the tests of strength in codes and laws become most severe during the bitter periods of human existence. Although somewhat distasteful as an undertaking, it occurs to the writer that turning the discussion of this code in the direction of certain subversive comments, born of a depression era, may serve the useful purpose of inviting further discussion from queer angles, to the ultimate end of making the code proof against as many honest subversive views as actually may be encountered.

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[37] Luke XIX: 12–25.

[38] Chairman, Dept. of Eng. Economics, The University of Wisconsin, Madison, Wis.

Standards of professional relations and conduct are always written with the young professional man, just emerging from school into his profession, particularly in mind. As it has been the writer's privilege to work quite intimately with many promising youngsters and to be in contact with them during some of their early struggles as junior engineers it seems particularly appropriate to try to discuss Professor Mead's paper as far as possible from their viewpoint.

As the period in the young engineer's life which the writer is trying to cover in particular might be called the "era of disillusionment," the approach to a code of ethics designed for his use will not be gentle. So true is this that only the successful engineer who is a living example of an ethical code can be sufficiently "strong medicine" to reach the aggressive young engineer passing through this era. For example, from the midst of a small circle of engineering students at the university not very long ago came this question, hurled like a hand grenade into an enemy trench: "Why don't they set up a practical course in engineering that will do us some good—a course in how to chisel the other fellow?" The questioner, during his Christmas vacation, had tried his hand for the first time in his father's business of selling motors and had lost his commission on an attractive sale through ignorance of the devious methods necessary (at least he was convinced that necessity was a major consideration) of getting around the Robinson-Patman Act. Words from his inexperienced fellow students or from a professor supposedly secure in his regular salary could not bring any soothing balm to this boy's wounds just then—only a father that played the game without winking at a Christian code of ethics or an eminent engineer in the game of selling, whose experience had proved that every turn in life's path must be navigated only by the light shed from the Golden Rule, could possibly make the crooked path for this boy for the time seem unattractive.

The foregoing incident reveals only too decidedly the need for the widespread endorsement of a living code and even more the dire need for successful engineers, through engineering societies and technical clubs, to get down among the young engineers and give from their very hearts something of their own devotion to the highest and best in professional practice in every devious professional path. It means also that, to make the proposed code a living code, the leaders in the fields of engineering selling and in the fields of engineering design and governmental practice, who have the status of regular employees of private corporations and governmental units, must rise to the occasion now and support the consulting engineer in the endorsement of this all-engineering code.

The need for a wide and adequate response from the successful engineering employee is brought out by the remark of another young engineer, employed in promotion and selling for one of the nation's largest trade associations, referring to this code and its author: "He never had the experiences of a struggling salesman. Any code is not going to reach us until it comes from someone whose experience parallels ours!" The justice of this criticism may be questioned, but the disillusioned young man attempts to be realistic, and to reach him codes must get down to where he is, not so much in content, but through the endorsement and living example of leaders in the very field in which the young man works.

In closing, the writer would like to make the comment that only widespread publication and discussion of a code can perfect it and fasten it into the minds and hearts of the profession. It seems to be a duty for some, who are in contact with special groups, to bring to the surface all manner of criticisms of this code which they find rising to them spontaneously from sincere interpretations of its contents. It stands as a monumental work. If the code successfully can stand trial in the fires of widespread discussion, the profession can have a code carrying the authority of an universal acceptance which can be expressed explicitly.

F. E. TURNEAURE,[39] HON. M. AM. SOC. C. E. (by letter).—Professor Mead has done a valuable piece of work in formulating in such detail the standards of professional relations and conduct which should be followed by members of the profession. The particular attention he has given to the student and the young engineer will be of especial value, not only to this particular group but also to those in the colleges who are concerned with their education.

To one who has been in contact with college students for many years and who has followed the later careers of many of them, it seems certain that such a detailed formulation of principles will be of great help to the beginner in reaching correct decisions in the analysis of his personal problems. Not infrequently the young engineer is confronted with situations involving questions of professional conduct that are somewhat difficult to analyze rightly and which, if wrongly answered, may lead him into further and more serious difficulties.

The increased attention being given this general subject in engineering schools is encouraging, but to be very effective the instruction should be given by men of wide experience as otherwise it is likely to be looked upon as too academic and idealistic to be of much practical value.

Opinions differ as to the quantitative significance of the percentage tables of characteristics quoted by Professor Mead but he has made it clear that the primary purpose of the quotations is to emphasize the vital importance of character and not to minimize the necessity for adequate technical knowledge. These two phases of the engineer's equipment are difficult to compare. They are both essential.

FRED ASA BARNES,[40] M. AM. SOC. C. E. (by letter).—It would be useless to attempt to add materially to this monumental paper; but the writer would like to subtract something from it. He cannot refrain from expressing his regret that Professor Mead has revived and republished the results of the so-called "investigation" of engineering education.[4] Of course, the author takes the pains to point out ("Introduction: Ability to Apply Principles to Practice") that "those factors common to educational requirements; * * * are absolutely essential to the success of the engineer," as do other writers who have made use of these data. For example, Howard Lee Davis in his book, "The Young Man in Business," states, "Of course, the employer cannot well place a man in a position for which he lacks the necessary education and experience." In the light of this comment, what do the results shown in Table 1 mean?

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[39] Emeritus Dean, Coll. of Mechanics and Eng., Univ. of Wisconsin, Madison, Wis.

[40] Prof., Railroad Eng., School of Civ. Eng., Cornell Univ., Ithaca, N. Y.

In the opinion of the writer they are the statistical outcome of the effort, probably unintentional, to mislead the 30,000 members of the Founder Societies into using their emotions in the rating of essential characteristics. That is, every one has a feeling, or emotion, that character and integrity are of great importance, and so they are; but what would one do if asked to "rate" his vital organs in "order of importance"? Is it not significant that less than one quarter of the engineers addressed replied at all and that replies from only a little over one sixth of them were sufficiently definite to be tabulated? It seems to the writer that, even if there were any logical and reasonable basis for rating essentials, fixing their relative importance to the nearest 0.1% (and apparently for all time) on the basis of so small a proportion of replies is fanciful in the extreme.

Furthermore, even if the results were valid, would they have much significance with reference to engineering education? Most thoughtful people realize the necessity of the characteristics mentioned in all lines of endeavor, but the writer would like to point out that students come to the colleges and universities with their characters and other personal traits rather thoroughly crystallized—that is, their development is largely the responsibility of the home and the preliminary schools. The college should and usually does continue the training along ethical lines, both by example and by formal courses, and Professor Mead's paper will long be a source of valuable material for such purposes.

C. A. Mead,[41] M. Am. Soc. C. E. (by letter).—The young engineer is likely to set aside the Code of Ethics adopted by the Society as applying to those in established private practice, even though the use of the word "client" is noted to include "employer." Professor Mead, in his timely paper, has expanded these rules so that all ranks and positions in the profession are covered in their application. They are not hastily conceived; and they carry conviction because the principles have long been proved and tried and have been found dependable and enduring, as every older man knows.

A former president of Hamilton College, in Clinton, N. Y., once defined character as "the habit of an attitude." Professor Mead rightly sets out character as the chief cornerstone in the career of any man and emphasizes the principles which are essentials in its formation. The use of the time a man makes of his "off" hours is a surer index to what he really is than those which are spent within his required hours of work. "Good measure, pressed down and running over" shows the manner of the man and will find recognition when the time for advancement comes.

The weight given to the characteristics necessary for the greatest success cannot be evaluated accurately, of course; but it is clear that all are agreed that character, judgment, and industry are three times as valuable as the knowledge of fundamental studies and technique, however valuable they are. These are matters on which the mind of the young engineer should be focused, and the earlier he assimilates them the sooner he will enjoy the freedom of success. They are the basic principles which a wise man cannot afford to ignore.

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[41] Upper Montclair, N. J.

This paper states positively what can be done by any man and does not leave the student with a skeleton of negative limitations; it gives him most valuable aid in a most comprehensive way. It is worthy of a place in the curriculum of every college, where it should be reviewed frequently so that it may become the mentor of every student.

As each man is the director of his own destiny, he should make the most out of his life. Precept and example are his only teachers at the outset of his career; but if he is careful in the selection of them he will find it easier in his later years, when ripe judgment is required, to give the honest and full return his clients expect.

Professor Mead's paper is replete with sound ideas of good conduct which, if followed, will lead to a success which is enduring and which will bring the maximum of satisfaction to him who makes it his rule of life. It is a most valuable contribution to ethical relations of professional engineers.

ADOLPH J. ACKERMAN,[42] M. AM. SOC. C. E. (by letter).—The engineers who can most appreciate Professor Mead's paper are those who, through a long record of high professional conduct, have experienced the many truths assembled in it.

As Professor Mead states in his "Introduction," the paper was written more particularly for the younger men of the profession. Each sentence might very well have been printed on a separate page because practically every one of them has enough substance to deserve a full page of discussion. The many principles are so fundamental that probably no one would attempt to take issue with them, yet how many really understand them, particularly among the younger and less experienced groups of engineers? The concept of "hard work" is quite different in the mind of the college student who is receiving all necessary financial support and enjoying the comforts of a well-furnished fraternity house, as compared with that of the student who is working his way by waiting on tables and doing miscellaneous chores, and who still succeeds in devoting enough time to intensive study to maintain a high scholastic standing. The same difference in concept applies to the many other professional ideals enumerated by Professor Mead.

Merely to hand out copies of Professor Mead's paper to students and young engineers is not enough. It would be unfair to expect of them a full understanding of the meanings and importance of those fundamental elements of character that go into a successful professional career. Such understanding can only be developed from personal contact with men of high professional standing who recognize their obligation of cultivating a closer master-apprentice relationship. The young engineer invariably visualizes his own career in terms of that demonstrated by some admired senior in his profession and is influenced to strive for the same high standards of conduct.

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[42] Director of Eng., Dravo Corp., Neville Island, Pittsburgh, Pa.

When tempted to forget fundamental principles, one is most readily reminded of them by experiences in which those principles stood the test. Men derive their courage from the courage demonstrated by their predecessors. The new generation of engineers is entitled to have the gospel of professional conduct preached to them by qualified ministers of the profession, using Professor Mead's paper as a text. His paper therefore, is not only a challenge to the young generation but also to the old.

Daniel W. Mead,[43] **Past-President and Hon. M. Am. Soc. C. E.** (by letter).—The kind expressions of approval of most of those who have discussed this paper are appreciated. The writer's object in preparing the paper, however, was for the purpose of drawing out discussion of certain principles so that a code of conduct might ultimately be issued which would, in general, meet the approval of a considerable body of the members of the Society rather than to offer in any sense a finished code. He therefore welcomes such criticism as has been offered and regrets that more detailed constructive criticism has not been proffered.

The paper is criticised: (1) Because it is wholly unnecessary; (2) because it is too long and involved; and (3) because it is not sufficiently inclusive.

With the first criticism the writer is not able to agree and will answer more completely later in the discussion. With the other two criticisms he must, in part, agree. To the man of wide experience who, during his life, has thoughtfully considered the various principles of conduct, even a very brief statement of principles may be all that is necessary, and even such a statement may be superfluous. For the inexperienced man, however, a code should be sufficiently explicit to cover most of the new relations which he will have to encounter during the development of his professional practice.

On the other hand, in presenting this subject for the consideration of the profession, it is practically impossible to cover all possible engineering relations, and it therefore becomes necessary to include only the most common and important relations with the belief that the examples given will serve as a basis for their intelligent extension to the unusual conditions which will occasionally arise.

Mr. Waters states: "It should be sufficient to say that 'every engineer should at all times think and act in accord with the highest principles of personal honor.'" It might also be suggested that the Golden Rule "Do unto others as you would have others do unto you" is all that is needed in such a code; and this is true in both cases if the individual is capable of applying either rule under the manifold conditions under which such rules should be applied.

This is the weakness of all brief rules of conduct. In themselves they contain no information as to how they should be applied in detail to professional conduct which, in the writer's judgment, is a serious mistake if they are offered to students or young engineers to guide them in their professional work. The writer believes that in the teaching of the application of the best principles of conduct the rule must be as specific as possible so as to apply as nearly as practicable to specific conditions.

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[43] Prof. Emeritus, Hydr. and San. Eng., Univ. of Wisconsin; Cons. Engr., Madison, Wis.

The writer agrees with Mr. Waters that "It is youth which must preserve the high ideals and standards of honor"; yet American schools are annually graduating thousands of young men from their engineering courses who have had no instruction in these high ideals and standards of honor or in professional conduct. Almost all universities, colleges, and schools are derelict in not affording instruction in this subject to all students.

The writer's experience is quite parallel with that of Professor McNew. Almost every year some young engineer from almost every school fails to abide by his agreement to accept a position offered at an early date and accepts instead a later offer at a higher salary. In this way the student's first action at the threshold of his professional life is unethical because of the financial advantage to him. This desire for financial gain is directly or indirectly the cause of most unethical conduct in professional work.

The fact that an older engineer of good reputation has not always acted in accordance with the principles of ethical conduct is not a sound basis for the similar conduct of others. The writer fears that there are no engineers in practice today who, looking back on their professional lives, can approve every act of their own professional conduct; and the personal acts which they now condemn are commonly the mistakes which they made, through ignorance or thoughtlessness, in the early days of their professional lives.

In answer to the comments of Mr. Howard, certain principles were repeated in more than one section in order to make each section more nearly complete in itself.

It will have to be admitted that the percentage method of evaluating "characteristics necessary to success in engineering" (Table 1) is inexact and faulty, but no other method seems available to illustrate the opinions of the profession as to the importance and essential value of attributes not commonly emphasized in engineering as now taught. Most professional men will agree as to the essential value of these characteristics if not in their percentage rating.

In reply to Mr. Consoer, the writer would call attention to Section IV, Paragraph 2, in which he states: "Honorable competition for promotion and opportunity for employment is an essential part of modern democratic civilization." The first sentence in Section IV, Paragraph 19, is simply an expression of the writer's idea as to the ideal way in which employment should be awarded. In the latter part of Paragraph 19 the writer expresses disapproval of unprofessional methods of competition which are all too common at the present time, and which he believes most practicing engineers also deprecate.

The writer was surprised at the principles advocated by Professor Peck. Such principles are unsound and dangerous, especially if presented to young men during the period when their professional and ethical attitudes are being formed. Professor Peck states that he is "opposed to fixed, rigid codes * * * be they religious, moral, or professional." Does his opposition include the legal codes of the federal and state governments?

Laws are but rules of conduct established by legislative action and enforced by the courts. These laws are supposed to represent the views of the majority of the people in the community in which they are established as to what individual action should or should not be allowed. There are thousands of these laws on the federal and state statute books; and there is scarcely one of them that under certain specific conditions may not be unfair and unjust to some individuals. Such laws frequently become archaic and are repealed or, if negligently left on the statute books, their absurdity becomes apparent and embarrassing, and public opinion demands and secures their repeal. The fact that the world is changing does not destroy the necessity for law, but it does necessitate its readjustment from time to time as conditions develop.

It is a fact with which every one is familiar that an individual may strictly observe the laws of the land and yet be an undesirable citizen and a poor neighbor. The idea that each individual can and should establish for himself rules of conduct for such relations as are not covered by law and without reference to the experience or opinions of others seems equally as absurd as would a similar attempt to establish principles of law. Laws must be established by the majority action of a legislative body, and rules of professional conduct must be based on the concurrent opinions of the members of a profession.

It is true of established rules of conduct, and also of rules of law, that few ethical or legal principles are universally applicable, and that in certain cases each individual must depend upon his common sense and conscience as to what his conduct should be under the limitations of the conditions under which his conduct must be exercised.

Common sense and conscience are the results of early training, of the personal influence of those with whom one comes in contact, of education, of experience, and of such reflection as the individual may give to these factors. On this account common sense and conscience are limited in their application to the breadth of the experience on which they are based, and can be applied successfully only within such limits. When that experience is extended, common sense and conscience are sometimes of benefit by analogy but are very often apt to be mistaken because of the limitation of experience.

A young man leaving college is plunged frequently into a life so different from his previous experience that he needs the advice of the men who have already had the experience that is about to become his. Rules and principles are then of value in reflecting the conclusions of those who have gone before. Just as precedent in the design and construction of engineering structures is of value when similar structures are being designed and constructed, so principles of conduct established by the experience of one's predecessors are of value in the consideration of one's own line of action. The writer will agree that precedents are not to be blindly followed; but to ignore the experience of others, either in engineering design or in conduct, is a dangerous and often a serious mistake. Regarding this point, Raymond Moley observes:[44]

> "What are principles that men live by? What, for that matter, is the meaning of principle itself? Reduce the question of principle to the case of an individual and one of the problems he faces. A man does not govern his life by chance. He learns, as the years pass, and profits by what he learns. He learns that there are some things that he cannot eat without distress. He learns that there are some games he cannot play. He learns

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[44] "Indispensable Principles," by Raymond Moley. An address delivered on Constitution Day—September 17, 1940—to the Union League Club of Chicago.

> that there are ways of doing his work better. He learns how to conduct his relations with other people. Out of the accumulation of individual experience he creates rules for himself. As time goes on, those rules become principles of living. He finds that by observing and respecting them he saves himself untold trouble and discomfort. He doesn't have to argue out thousands of individual decisions with himself. He depends upon his principles. Ultimately he lives not only with but by them."

It is unlikely that a man of advanced age and long experience would, in his ordinary relations in life, find it necessary to study a code of conduct in order to determine what his own conduct should be; neither would he ponder very deeply concerning the various possible outcomes of his action. His answer would be given at once, based on his established principles, and would probably be correct. If, however, the conduct concerned new conditions entirely beyond and different from his previous experience, then comes the necessity for due consideration; and in such cases the opinion of those who have had similar experiences and have reached definite conclusions cannot safely be ignored.

A person does not seek legal advice concerning his ordinary conduct in everyday life, but when he enters into new legal relations with which he is not familiar it is the part of wisdom to seek advice.

The Golden Rule is, and should be, in general, the basis of almost all ethical conduct. The Golden Rule is as sound today as it was in the days of Christ. It may puzzle the philosopher who desires to live by the Golden Rule as to what he should do if he captures a thief who is attempting to plunder his home. However, the practical man who also desires to live by this rule would recognize his obligations to society and would at once hand the malefactor over to the police.

So far as the writer's information goes, a code of ethics was first established for physicians about 400 B.C. by Hippocrates, a famous Greek physician often called "The Father of Medicine"; and the writer understands that Hippocrates' "oath of service" is still used in some medical schools. This would seem to indicate that a "changing world" has not seriously affected the code of Hippocrates in more than twenty-three hundred years. In a similar manner, although certain purely technical requirements of a code of conduct may change with changing conditions, the fundamental principles of good conduct and good ethics are unchangeable and eternal.

Legal ethics is of many years standing. Today thousands of codes have been adopted by professional societies, technical and business organizations and associations, and numerous books have been published discussing the basis of business and professional relations contained, or those which should be contained, in these codes. Is the assumption warranted that all this effort is wasteful and unnecessary and that each individual in all these diverse interests is qualified to evolve from his own reflections all of the principles of conduct which he should exercise under each of the thousands of circumstances in which he may happen to be or into which he may possibly enter from an entirely different environment? The writer cannot but feel that such a proposition is an absurdity.

Due consideration of what is best for all is both desirable and essential. However, the writer has never seen the "edge of the moral responsibility" of any man so keen that he thought it would be dulled by the reading of a code of conduct or by consulting the opinion of those who have had previous experience in an unknown field into which the individual was about to enter, whether such field be in engineering or in conduct.

There is no doubt that there are many who are mentally incompetent and that in many cases failures are due to such cause. However, lack of ambition and mental laziness are commonly ascribed to such cause. If and when psychological findings become uniformly reliable, it will perhaps become desirable to limit the efforts of students to those fields within their abilities. In the meantime, however, the likes and dislikes and the ambitions of the student are the general guide to his line of efforts and he should not be discouraged by imaginary inhibitions, etc.

The writer can appreciate the experience of Prof. E. D. Ayres with his students. In many years in teaching Engineering relations to senior engineers, the most common inquiry received by the writer was perhaps how to do an unethical thing in an ethical manner. The action usually involved was the breaking of an agreement to accept employment, which agreement was made months before graduation, in favor of accepting a more remunerative job offered just prior to graduation. Although the writer never was able to give an answer satisfactory to the student, he always advised such students—if they had decided to break their first engagement—at least to request release from the earlier commitment as a more manly course than bluntly advising his first would-be employer that he had taken another and better job. The question asked Professor Ayres as to "how to chisel the other fellow" has been commonly answered in the words of David Harum: "Do the other fellow the way he would do you, and do him first." This has been applied not only in horse trading but also in other lines of business; but it is far from good ethics for the business man, and especially for the professional man.

In closing this discussion the writer desires to state that this paper was prepared for three separate but similar reasons.

First: In his college work the writer does not remember that he ever received any material information or instruction on Professional Relations or Professional Conduct. He was obliged to gather such information in the expensive school of experience and "hard knocks." He has always felt that he might have done much better work and made many less errors if there had been available during that period the experience and conclusions of those who had already trod the difficult road to successful professional practice. He has felt, therefore, that it was incumbent upon him to make available to the younger men of the profession the conclusions which he has reached after more than fifty-six years of professional practice.

Second: Early in his professional practice the writer's attention was drawn to the unfortunate reaction of certain young men who, after leaving college, were thrown under the influence of certain men whose practice was never to subordinate profit to high ideals. He witnessed the fact that under such influence these young men seemed to assume that the actions they observed must be common to their profession; and such actions they too often adopted in their own conduct. The writer then believed, and still believes, that some advice given during college training of these young men would in many cases have afforded a means of avoiding the mistakes made by them in the early days of their experience. For this reason, when the writer became a teacher in an engineering school, he gave some time, in all the courses which he taught, to instruction covering the proper engineering relations in professional life; and in his classes in Contracts and Specifications the writer spent considerable time discussing this important subject. The reaction of his students who have volunteered their opinion of his efforts, after many years in practice, has convinced him that at least in many cases the discussion of these subjects has been of material benefit to many engineering graduates.

Third: During his experience as a teacher, the writer had occasion to read many professional codes of ethics and numerous books written on the subject, in order to secure proper material for the presentation of this subject to his classes. He found that most adopted codes had evidently been written for older professional men in general practice and covered only a very limited field for the younger men who occupied the lesser positions in the engineering field. The writer is thoroughly convinced that, if an advisory code of professional conduct is available and is called to the attention of the younger men during the formative period of their professional training, it will be of material assistance to many of them. He believes that most engineering students desire to do the right thing and will, in their professional lives, follow the habits which they have formed in their youth in playing their college games, of following the rules of the game in playing the most important game of life.

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GENERAL WEDGE THEORY OF EARTH PRESSURE

By Karl Terzaghi,[1] M. Am. Soc. C. E.

With Discussion by Messrs. Howard F. Peckworth, Jacob Feld, M. G. Spangler, Donald M. Burmister, and Karl Terzaghi.

Synopsis

During the twenty years since the first World War the most important development in earth pressure theory has been the transition from the original theory of Coulomb to a broader concept called "the general wedge theory."

According to this theory the shape of the surface of sliding and the position of the resultant earth pressure are a function of the degree and type of freedom for lateral expansion of the supported earth. Coulomb's theory represents a special case of the general wedge theory. Its validity is limited to the pressure of earth on supports such as retaining walls whose lateral resistance is independent of their amount of yield. In every other case the general wedge theory should be used. The only condition required to make this theory valid is an almost complete mobilization of the shearing resistance of the supported earth within a continuous zone extending from the foot of the bank to its upper surface. This single requirement, however, is sufficient to exclude the general wedge theory (or any other theory involving the condition of incipient failure) from application to the pressure of soft clay or other highly elastic types of soil on the timbering of cuts or tunnels.

General Principles

The essentials of the general wedge theory can be described as follows: If the lateral support of a mass of earth yields, a wedge-shaped body of earth descends toward the yielding support. Before this process begins, however, the weight, W, of the wedge (see Fig. 1), the resultant reaction, F, along the surface of sliding, and the reaction P_a, acting along the back of the lateral support, must be in equilibrium, and the shearing resistance of the backfill must be almost completely mobilized within a continuous zone, extending from the

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Note.—This paper was presented at the meeting of the Soil Mechanics and Foundations Division at Rochester, N. Y., on October 14, 1938, and published in October, 1939, *Proceedings*.

[1] Dr. Ing., Lecturer, Harvard Univ., Cambridge, Mass.

foot of the backfill to its upper surface. This broader concept of the general wedge theory involves no assumption concerning the shape of the surface of sliding and none concerning the location of the center of the resultant earth pressure. The surface of sliding must satisfy the condition that the corresponding reaction, P_a, required to prevent the descent of the wedge, shall be a maximum and that the yield of the lateral support shall be consistent with the conditions of stress and strain within the wedge. An analysis of the manner in which the lateral support yields, and the influence of this yield on the conditions of stress and strain in the wedge, was presented by the writer in 1936.[2] This analysis led to the conclusion that the shape of the surface of sliding and the position of the resultant earth pressure are a function of the degree and type of freedom for lateral expansion of the supported earth. Depending on this degree of freedom, the surface of sliding can assume any shape intermediate between an inclined plane (Fig. 1(*a*)) and a conchoidal surface approaching a vertical asymptote (Fig. 1(*d*)). The center of the resultant earth pressure may be anywhere between 0.33 h and $> 0.5\ h$, and the corresponding distribution of the lateral pressure ranges between hydrostatic and roughly parabolic, as shown in Fig. 1.

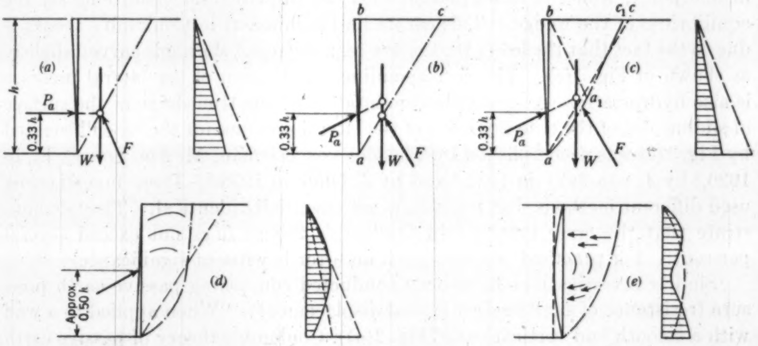

FIG. 1

In the general wedge theory, if the supplementary assumptions are introduced that the surface of sliding is plane and that the distribution of the pressures over both the back of the wall and the surface of sliding is hydrostatic, the result is the well-known earth pressure theory of Coulomb. Analogous to the introduction of these assumptions in the general wedge theory is the assumption that plane sections remain plane, which Navier introduced in the general theory of bending to obtain the simplified theory of bending. If this simplified theory is applied to the bending of short and very thick beams, erroneous results are obtained. Similarly, the theory of Coulomb leads to unsatisfactory results if the conditions of the problem are incompatible with the fundamental assumptions.

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[2] "A Fundamental Fallacy in Earth Pressure Computations," by Karl Terzaghi, *Journal*, Boston Soc. C. E., April, 1936.

In order to define the boundaries between the realm of validity of Coulomb's original theory and that of the general wedge theory, the writer presents herein an analysis of the different conditions that are likely to influence the intensity and the distribution of the active earth pressure. For the sake of brevity the analysis is limited to the investigation of pressures exerted by cohesionless backfills on vertical lateral supports. Neither the influence of surcharges nor that of percolating water on the lateral pressure will be taken into consideration.

WALLS TILTING AROUND INNER LOWER EDGE

If the back of a wall is perfectly frictionless and if, in addition, the wall yields by tilting around its lower edge, Coulomb's assumptions are fully satisfied and the results obtained by his method are correct. This case is illustrated by Fig. 1(*a*). If the back of the wall is rough, involving an angle of wall friction δ, the forces that act on the wedge according to Coulomb's theory are shown in Fig. 1(*b*), and the corresponding distribution of the lateral pressure over the back of the wall is hydrostatic. The forces shown in Fig. 1(*b*) fail to intersect in one point, which is incompatible with the fundamental conditions for the equilibrium of the wedge. This contradiction inherent in Coulomb's theory is due to the fact that the lower part of the real surface of sliding is curved slightly as shown in Fig. 1(*c*). The corresponding distribution of the lateral pressure is also hydrostatic. An accurate computation of the true shape of the surface of sliding and of the true intensity of the lateral pressure for the case illustrated by Fig. 1(*c*) was accomplished by Theodor von Kármán, M. Am. Soc. C. E., in 1926,[3] by J. von Jaky in 1931,[4] and by J. Ohde in 1938.[5] These investigators used different methods, but the results are essentially identical. They demonstrate that the error involved in Coulomb's theory does not exceed several per cent. For practical purposes such an error is without significance.

Similar investigations have been conducted concerning passive earth pressure (resistance of sand against lateral displacement). When applied to a wall with a smooth and vertical back (Fig. 2(*a*)), Coulomb's theory of passive earth pressure is strictly correct. On the other hand, if the theory is applied to a similar wall with a rough back, as shown in Fig. 2(*b*), Coulomb's assumptions concerning the forces that act on the wedge are conspicuously in contradiction to the conditions for the equilibrium of the wedge. Both experiment and the advanced theories revealed by von Kármán[3] and von Jaky[4] demonstrate that the surface of sliding is curved as shown in Fig. 2(*c*). They also demonstrate that the error due to Coulomb's assumptions may exceed 30%, provided the value of the angle of wall friction δ approaches that of the angle of internal friction, ϕ. Hence, if δ exceeds several degrees, it is necessary to modify the theory by assuming that the sliding surface is not a plane. For all practical purposes, one can approximate the shape of the sliding surface by considering the lower portion as a segment of a logarithmic spiral and the upper portion as a straight line rising at an angle of $(45 - 0.5\,\phi)°$ to the horizontal, as shown in Fig. 2(c).[5] The position of the center of the spiral must be determined in such a manner that the corresponding value of the passive earth pressure is a minimum. The most convenient method for accomplishing this result is by trial. An accurate method of determining the shape of the surface of sliding was introduced in 1938 by Ohde.[5] However, the application of this method requires considerable time and labor, and the difference between the results thus obtained and those obtained by means of the aforementioned approximate method does not exceed a few per cent. The assumption of a hydrostatic pressure distribution is reasonably correct, provided the wall advances toward the fill by tilting around the lower edge.

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[3] "Über elastische Grenzzustände," by Theodor von Kármán, *Proceedings*, Second International Cong. for Applied Mechanics, Zurich, 1926.

[4] "Die klassische Erddrucktheorie. Vezügyi Közlemenyek," by J. von Jaky, Budapest, 1931.

[5] "Zur Theorie des Erddruckes," by J. Ohde, *Die Bautechnik*, 1938.

Far more important than the errors due to the approximation regarding the shape of the surface of sliding are those due to the uncertainties concerning the angle of wall friction. Thus far the results of pertinent investigations seem to indicate that the angle between the resultant passive earth pressure and the normal to the wall is considerably smaller than the angle of friction between backfill and wall unless the state of stress in the mass of sand subject to the later pressure approaches the point of failure. Due to the important bearing of the subject of passive earth pressure on the stability of bulkheads, further experimental investigations concerning the angle of wall friction are urgently needed.

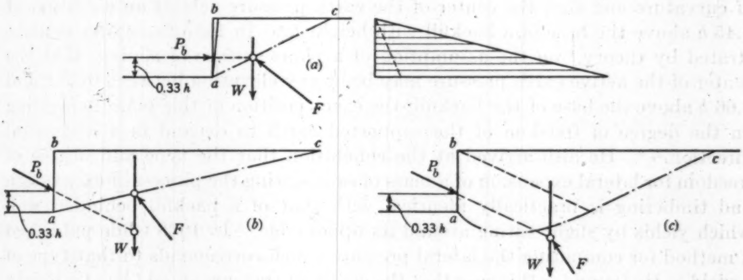

FIG. 2

In the cases illustrated by Fig. 1(a) to 1(c) the distribution of the active earth pressure over the back of the wall is hydrostatic. The theories which apply to these cases may also be used for computing the active earth pressure on retaining walls or other lateral supports which are capable of yielding indefinitely without being weakened. Large-scale earth pressure tests made by the writer in 1929 demonstrated conclusively that the distribution of the lateral pressure of sand is always hydrostatic after the lateral support has yielded beyond a certain critical distance which depends on the height of the fill and on the character of the backfill.[6]

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[6] "Large Retaining Wall Tests. I. Pressure of Dry Sand," by Karl Terzaghi, *Engineering News-Record*, February 1, 1934.

On the other hand, the capacity of the timbering in a cut to yield under pressure is very limited and in no case can it be expected to reach the value of the yield required to induce a hydrostatic pressure distribution. For this reason it is necessary to consider the influence of a small yield of a lateral support under the conditions associated with excavation and timbering on the intensity and the distribution of the lateral pressure.

TIMBERING OF EXCAVATIONS IN SAND

As early as 1908, the late J. C. Meem, M. Am. Soc. C. E., on the basis of his experiences in subway construction in New York City, maintained that the distribution of the earth pressure over the timbering of cuts is decidedly non-hydrostatic, and that the surface of sliding intersects the surface of the bank at a right angle.[7] His statements were corroborated by similar practical experiences described in 1920 by H. G. Moulton,[8] M. Am. Soc. C. E. While making his large-scale earth pressure tests in 1929 the writer observed that the surface of sliding in the backfill of a sliding wall has an abnormally small radius of curvature and that the center of the earth pressure acts at an elevation of 0.45 h above the base of a backfill with height h.[6] In 1935 the writer demonstrated by theory,[9] on the assumption of a plane surface of sliding, that the center of the active earth pressure may be at any elevation between 0.33 h and 0.66 h above the base of the backfill, the exact position of this point depending on the degree of freedom of the supported earth to expand in a horizontal direction.[2, 9] He also arrived at the conclusion that the type and degree of freedom for lateral expansion of a mass of sand during the process of excavation and timbering is practically identical with that of a backfill behind a wall which yields by slight tilting around its upper edge. In 1938 Ohde published a method for computing the lateral pressure which corresponds to that type of a yield. He found for this case that the center of pressure should be at a height of 0.55 h above the base of the fill and that the lateral pressure is appreciably greater than the Coulomb pressure. The last mentioned fact can be expressed by an equation

$$P_{an} \doteq C_k \, k_a \frac{w_m h^2}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which P_{an} = horizontal component of the total lateral pressure; w_m = unit weight of the backfill; h = depth of the cut; k_a = hydrostatic pressure ratio for Coulomb conditions; and C_k = the wedge factor, a factor that expresses the influence of the state of strain in the wedge on the total intensity of the lateral pressure.

The value of k_a can be computed by means of Coulomb's theory. It depends only on the angle of internal friction ϕ, and on the angle of wall friction, δ. The wedge factor C_k depends on the state of strain imposed on the wedge by the lateral boundary conditions. It can be computed by means of Ohde's theory, which is based on the assumption that the surface of sliding can be replaced by an arc of a circle. However, the method is far too complicated for universal practical application. For this reason the writer developed a simplified method of computation. It is based on the empirical facts that the surface of sliding intersects the horizontal surface of the bank at a right angle and that the center of the lateral pressure on the timbering of a cut with a depth h in sand is somewhere between an elevation of 0.50 h and 0.60 h above the bottom of the cut. The agreement between Coulomb's and von Kármán's theory of the active earth pressure and between Ohde's rigorous and simplified theory of the passive earth pressure prove that a moderate departure of the assumed shape of the surface of sliding from the real one has little influence on the results. In the case under consideration the writer found that the most convenient substitute for the real surface of sliding is a logarithmic spiral with the polar equation

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[7] "The Bracing of Trenches and Tunnels, with Practical Formulas for Earth Pressures," by the late J. C. Meem, *Transactions*, Am. Soc. C. E., Vol. LX, June, 1908, p. 1.

[8] "Earth and Rock Pressure," by H. G. Moulton, *Transactions*, Am. Inst. of Mining and Metallurgical Eng., 1920.

[9] "Distribution of the Lateral Pressure of Sand on the Timbering of Cuts," by Karl Terzaghi, *Proceedings*, International Conference on Soil Mechanics and Foundation Eng., Vol. I, Cambridge, Mass., 1936.

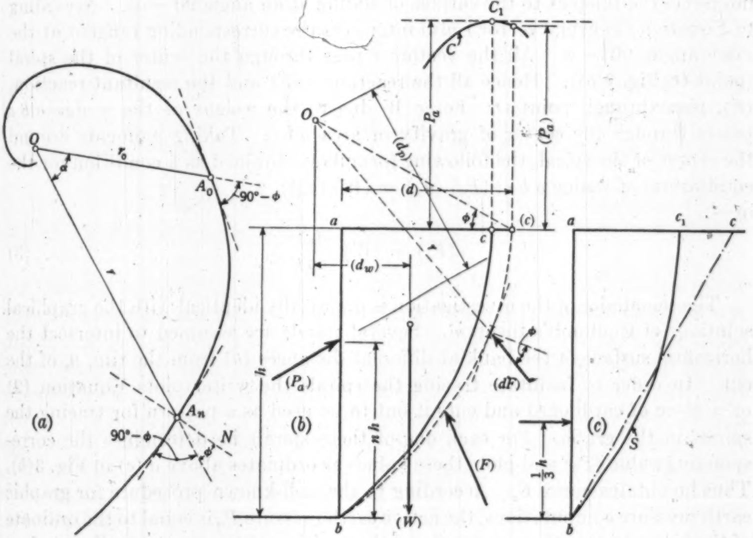


FIG. 3

$$r = r_0\,\epsilon^{\alpha \tan \phi} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

in which r = length of a vector through the center of the spiral; r_0 = length of an arbitrarily selected reference vector; ϵ = base of natural logarithms; α = angle between r and r_0, in radians; and ϕ = angle of internal friction of the backfill. Fig. 3(a) shows such a spiral corresponding to Equation (2). It has the property that a vector r passing through the center of the spiral and any arbitrary point A_n of the spiral intersects the tangent to the spiral in point A_n at an angle $90 - \phi$. As a consequence the angle between the vector r and the normal N to the tangent in point A_n is equal to the angle ϕ of internal friction.

In order to satisfy the boundary conditions of the problem, the spiral is oriented with reference to the face of the bank $a\,b$, Fig. 3(*b*), in such a manner that the spiral passes through the foot of the bank b, Fig. 3(*b*), and that it intersects the horizontal surface of the bank at a right angle, at an arbitrary distance (d) from the upper rim a. The angle of wall friction, δ, and the elevation $n\,h$ of the center of pressure are assumed to be known. The general wedge theory requires that the frictional resistance of the surface of sliding should be almost fully mobilized over the entire surface of sliding, $b\,c$ in Fig. 3(*b*). As a consequence the reaction, dF, on each element of the sliding surface in Fig. 3(*b*) intersects the tangent to the surface of sliding at an angle $90 - \phi$. According to Equation (2) every vector r also intersects the corresponding tangent at the same angle, $90 - \phi$. All the vectors r pass through the center of the spiral (point O, Fig. 3(*b*)). Hence all the reactions (dF) and the resultant reaction, (F), pass through point O. Force W due to the weight of the wedge $a\,b\,c$ passes through the center of gravity of area $a\,b\,c$. Taking moments around the center of the spiral, the following formula is obtained as a condition for the equilibrium of wedge $a\,b\,c$: $(P_a)\,(d_p) = (W)\,(d_w)$;
or—

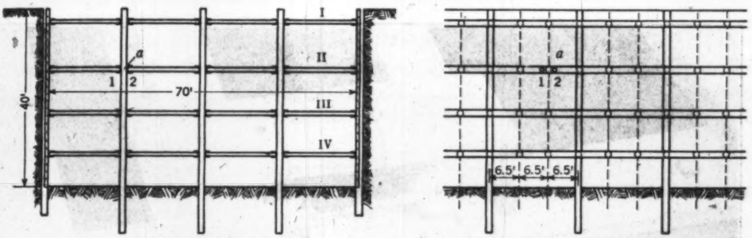
$$(P_a) = (W)\,\frac{(d_w)}{(d_p)} \quad \text{.......................}(3)$$

The remainder of the investigation is practically identical with the graphical solutions of Coulomb's theorem. Several spirals are assumed to intersect the horizontal surface of the bank at different distances (d) from the rim, a, of the cut. In order to facilitate tracing the spirals the writer plots Equation (2) on a piece of cardboard and cuts it out to be used as a pattern for tracing the spirals in the graph. For each one of these spirals he determines the corresponding value (P_a) and plots these values as ordinates above a (c) in Fig. 3(*b*). Thus he obtains curve C. According to the well-known procedure for graphic earth pressure computations, the active earth pressure P_a is equal to the ordinate of the highest point C_1 on curve C and the position of the corresponding surface of sliding is determined by point c beneath point c_1.

One would expect the greatest error involved in the graphical method illustrated by Fig. 3 to occur when the method is applied to the computation of the active earth pressure on a wall, Fig. 1(*b*), which yields by tilting around its lower edge, because in this extreme case the surface of sliding does not resemble a logarithmic spiral. The corresponding numerical values are $n = \frac{1}{3}$ and the wedge factor $C_k = 1.0$. By introducing into his computation the values $\tan \phi = 0.6$ and $\delta = 0$, the writer obtained $C_k = 0.94$ instead of the correct value 1.0. Curve S in Fig. 3(*c*) represents the curved sliding surface for this case and line $b\,c$ is the corresponding surface of sliding obtained by means of Coulomb's theory. In spite of the conspicuous difference between these two lines, the error does not exceed 6%. If the value of n is high, the surface of sliding actually has the shape assumed in the general wedge theory. Therefore the error can be expected to be still smaller. For $\phi = 37°$; $\delta = 26°$; and $n = 0.6$ the computation furnished $C_k = 1.27$ and for $n = 0.50$, $C_k = 1.18$. In general the value C_k increases with increasing values of n, ϕ, and δ.

FIELD MEASUREMENTS

In order to check the writer's theory as it applies to the lateral pressure exerted on the timbering of excavations, the Siemens Bau Union, in 1936, measured the pressures in the struts of a 38-ft subway cut in Berlin, Germany. The sides of the cut consisted mostly of sand and fine gravel. Fig. 4(*a*) shows a cross section through the cut, and Fig. 4(*b*) shows a longitudinal section. Before excavation was begun, two outer and three inner rows of I-beams were driven into the ground. In the outer rows the spacing was 6.5 ft, and in the inner rows it was about 20 ft. As excavation proceeded, horizontal boards were inserted between the flanges of the steel beams of the outer rows. The struts were arranged so as to brace the steel beams of the outer rows against one another across the cut. After excavation was finished the lateral pressure of the earth was taken by four sets of struts. In every set the average horizontal distance between the struts was 6.5 ft. The vertical distance between the sets decreased from the top set to the bottom set. Each strut consisted of four pieces bearing against the web of horizontal channels and fastened to the steel I-beams of the inner rows. The distance between the webs of each pair of channels was maintained by means of wooden spacers (*a*, in Fig. 5).



(*a*) CROSS SECTION (*b*) LONGITUDINAL SECTION

FIG. 4

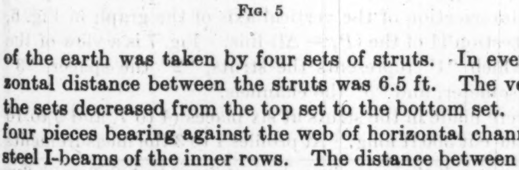


FIG. 5

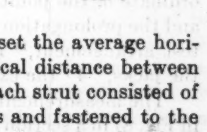


FIG. 6

In order to measure the axial pressure in a strut, two small hydraulic jacks were inserted between the channels (Fig. 5(*b*)) on both sides of the spacer. Then the jack pressure was increased gradually and the corresponding increase, Δl, of the distance l, between points 1 and 2 was measured. The relation between the total jack pressure, P_j, and the corresponding values of Δl are shown in Fig. 6. As long as the pressure exerted by the jacks is less than the pressure in the strut, the jacks merely relieve the pressure in the spacer without changing the pressure in the long sections of the strut.[10] The corresponding increase of the distance between points 1 and 2 is slight, as shown by the steep rise of section I of the $(P_j - \Delta l)$-line in Fig. 6. However, as soon as the jack pressure exceeds the pressure in the struts, the pressure in the long sections of the strut increases and, consequently, the deflectometer readings increase rapidly (see flat section II of the $(P_j - \Delta l)$-line in Fig. 6). The pressure that acted in the strut before the jack pressure was applied is determined by the ordinate of the point of intersection of the vertical axis of the graph in Fig. 6, and the prolongation of section II of the $(P_j - \Delta l)$-line. Fig. 7 is a view of the test arrangement,[10] in which "1" represents the struts, "2" the spacer, "3" the jacks, "4" the extensometer, and "5" the channels.

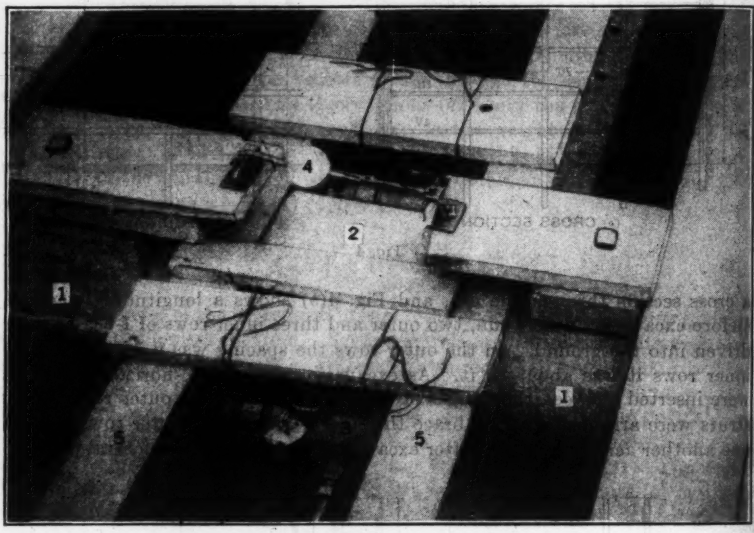


Fig. 7

The measurements were made in the struts at six places (4 to 7, and 9 to 10 in Fig. 8) in a section of the cut 360 ft long. At profiles 1 to 3 the measurements were limited to the top strut and at profile 8 to the second strut from the top. Fig. 9 shows the results of some of the measurements.

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[10] "Mitteilung über die Messung der Kräfte in einer Baugrubenaussteifung," by A. Spilker, *Die Bautechnik*, 1937, Heft 1.

In order to discover whether the frictional resistance of the supported banks had been mobilized, a comparison was made by the writer between the theoretical value and the measured value of the total pressure in the struts at the different sections. For this purpose, Equation (1) was used. The unit weight of the sand[10] was 1.7 tons per cu m, the angle of internal friction ϕ was 37°, and the angle of wall friction δ was 26°. The values of the angles ϕ and δ undoubtedly were estimated and not measured. Due to the absence of fine material, the cohesion was negligible.

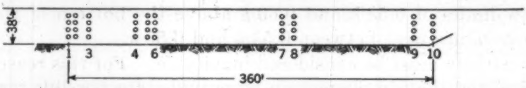

FIG. 8

With the numerical values mentioned, Coulomb's theory yields the value $k_a = 0.204$; and, from the general wedge theory, $C_k = 1.22$. The computations, according to the general wedge theory, yield a theoretical value for the total lateral pressure which is an average of 25% greater than the value obtained from the measurements. These findings seem to indicate that the angle of internal friction of the supported material was appreciably greater than the estimated value of 37° (at least 41°), and that the shearing resistance of the supported banks was almost fully mobilized, satisfying the only fundamental requirement of the general wedge theory.

According to the general wedge theory for the pressure of sand on timbering in excavations, the distribution of the pressure over the lateral support should be more or less parabolic, having a maximum pressure at approximately half the depth of the cut. This conclusion is fully substantiated by the results of the measurements shown in Fig. 9.

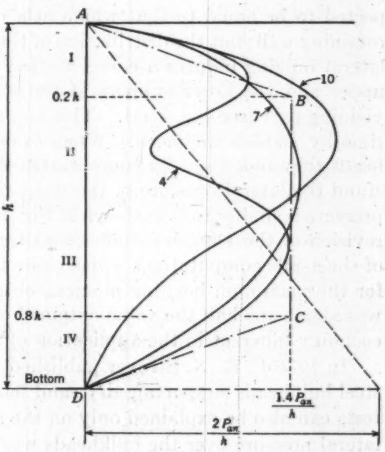

FIG. 9

The data shown in Fig. 9 also indicate the magnitude of the deviations of the true pressure distribution from the statistical and theoretical average. According to the geological profile of the cut, the supported banks are fairly uniform. Hence, it seems most likely that the observed variations are essentially due to accidental variations in the procedure followed in constructing the timber work.

Foremost among these variations is undoubtedly the variation in the force with which the wedges were driven. Another cause may be the local variations in the density of the supported material, because such variations are likely to induce local arching. This might account for the unusual shape of pressure curve 4 in Fig. 8. On the other hand, the location of the center of the lateral earth pressure was quite remarkably consistent. It varied between the relatively narrow limits of 0.53 h and 0.60 h above the bottom of the cut. The most frequent values were between 0.53 h and 0.55 h.

These variations must be considered inevitable. For this reason the writer has always insisted that ample margin be provided for possible scattering. In Fig. 9 this margin is indicated by the trapezoid $A\ B\ C\ D$. If h indicates the depth of the cut and P_{an} the horizontal component of the total lateral pressure on the timbering, the width of the trapezoid is equal to $1.4 \frac{P_{an}}{h}$. Fig. 9 shows that the observed variations are at least as large as those which are taken care of by the suggested margin.

Flexible Bulkheads

Another problem of outstanding practical importance concerns the earth pressure exerted on flexible bulkheads. The total earth pressure can be expected to be equal to that which acts on the back of a yielding, non-flexible retaining wall; but the distribution of the pressure is quite different, because the lateral bulging induces a concentration of the pressures in the vicinity of the upper and the lower support at the expense of the pressure on the middle, yielding part (see Fig. 1(*e*)). This fact seems to have been noticed for the first time by Danish engineers. When examining some of their century-old bulkheads they found out by computation that these bulkheads could not possibly stand the lateral pressure of the supported earth unless the distribution of the pressure is similar to that shown in Fig. 1(*e*).[11] This observation led to a radical revision of the Danish specifications for the design of bulkheads.[12] In a case of sheet-pile computations which came to the attention of the writer, the value for the maximum bending moment obtained by means of the Danish method was about one half the value obtained on the basis of Coulomb's theory. The economy inherent in the application of this procedure is obvious.

In 1936 P. R. N. Stroyer published the results of experiments with flexible steel bulkheads supporting dry sand backfills 3 ft deep.[13] The results of these tests can also be explained only on the assumption that the distribution of the lateral pressure over the bulkheads was such as that shown in Fig. 1(*e*), which in turn requires intense "arching" within the sand behind the bulkhead. Some engineers still seem to believe that the arching effect is a temporary phenomenon, destined to disappear in time. The Danish experience mentioned previously represents one of many empirical facts which invalidate this pessimistic opinion. Some of the Danish bulkheads were exposed for centuries to the pounding of waves, and for decades to heavy traffic, without the distribution of the lateral pressure becoming "hydrostatic."

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[11] "Calcul des murs de quai," by G. Schønweller, *Proceedings*, World Eng. Cong., Rept. No. 416, Tokyo, 1929.

[12] "Forelobige Regler for Beregning af Jaernbetonkonstruktioner i Vandbygning," published by the Dansk Ingeniörforening (no date); see also "Ztschr. des ständ. Verb. intern. Schiffahrtskongr.," No. 7, January, 1929.

[13] "Earth Pressure on Flexible Walls," by P. R. N. Stroyer, Paper No. 5024, *Journal*, Inst. C. E., November, 1935 (discussion of this paper by the writer, January 1936).

The arching between the upper and the lower support of flexible bulkheads eliminates Coulomb's theory from application to bulkhead problems. The influence of the lateral deflection on the location of the center of pressure is not yet known. Undoubtedly, it depends not only on the type of backfill and on the flexibility of the wall, but also on the distance through which the upper and the lower supports of the bulkhead are capable of yielding and on the density of the backfill.

Because of the outstanding practical importance of the problem, the writer advised the Soil Mechanics and Foundations Division of the Society to include it in its research program. The variables to be considered in such an experimental investigation are: (*a*) Degree of fixation of the lower part of the bulkhead, (*b*) position of the upper support (anchorage) with reference to the top of the backfill, (*c*) yield of the anchorage, (*d*) deflection of the flexible part of the bulkhead with reference to a plane through the upper and lower supports, and (*e*) the relative density of the backfill. Investigations of this kind could be made in a well-equipped earth-pressure laboratory at a small expense for additional special equipment. It would also be advisable to include in the program the measurement of the deflection of full-sized bulkheads, to compute the corresponding bending moments and to supplement the results of the measurements by determining the physical properties of the backfill.

PRESSURE EXERTED BY COHESIVE EARTH

For cohesive soils the general wedge theory is based on Coulomb's equation for the shearing resistance, s, of cohesive soils. In this empirical equation,

$$s = c + p \tan \phi \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

c is Coulomb's cohesion value, p is the unit pressure normal to the surface of sliding, and ϕ is Coulomb's angle of internal friction. However, there are many conditions under which Equation (4) does not give even approximate values. Hence, at the very best, it should be considered a crude approximation to reality. If used without careful discrimination, it can be seriously misleading.[14] For cohesive soils, the influence of the type of yield of the lateral support on the location of the center of pressure, and on the distribution of the lateral pressure, is still more important than it is for cohesionless ones. If the lateral support yields by tilting around its lower edge, one must anticipate a hydrostatic pressure distribution, beginning at some depth below the upper edge. Above this depth the soil can stand without any lateral support. On the other hand, if the lateral expansion of the wedge increases with depth (as it does during the excavation of a timbered cut) the heavy pressure acts on the upper part of the timbering, whereas the lowest part of the sides can be left without any support. These facts can be demonstrated easily by means of the theory of arching.[9] By analogy with reference to the theory of cohesionless soils, one may also conclude, in accordance with experience, that the section through the surface of sliding should start with a vertical tangent at the surface of the bank and descend toward the foot of the bank with a very marked curvature. Hence, there is no contradiction between the general wedge theory and experience, provided the yield of the lateral support is sufficient to mobilize the major part of the shearing resistance of the supported banks.

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[14] "Einfluss des Porenwasserdruckes auf den Scherwiderstand der Tone," by Karl Terzaghi, *Deutsche Wasserwirtschaft*, 1938, Heft 8/9.

This mobilization of the shearing resistance, however, can only be anticipated in comparatively dense strata of sandy soils. If the soil is fairly elastic, as is the case for plastic or soft clay, the lateral expansion of the sides of the cut during excavation and timbering can scarcely be expected to result in mobilizing more than a fraction of the shearing resistance of the supported material. In this case the intensity of the lateral pressure of the earth must be greater than the value obtained by means of the general wedge theory. The importance of the difference depends on the elastic properties of the supported soil, the distance through which the material can yield during the process of excavation and timbering, the state of stress that existed in the earth before excavation was begun, and on the time factor. Because of the difficulties connected with evaluating these many variables, and the complicated elastic properties of cohesive soils, the prospects of predicting the lateral pressure successfully by theory are slight. A substantial advancement of pertinent knowledge can be expected only from actual measurement of the pressure in the braces in cuts through different clay soils, and subsequent correlation of the results with the physical properties of the materials encountered in the cuts in which the measurements were made. A satisfactory method for making such measurements is described herein, and illustrated by Figs. 5 to 7. If steel beams are used to support the sides of a cut, the pressure in the struts can be determined by means of a strain gage such as the Whittemore strain gage. The reference points should be placed and their distance recorded before the beams are lowered into the cut.

On account of the appalling scarcity of published information concerning the pressure of clay on timbering in excavations, the results of every reliable observation regarding this subject should be considered important. Therefore the writer has requested the Soil Mechanics and Foundations Division to collect such data and to promote the measurement of the pressure on the timbering in excavations wherever there is an opportunity to do so. The results thus obtained could not fail to produce a radical revision of the specifications for the computation of earth pressure under such conditions.

DISCUSSION

Howard F. Peckworth,[15] M. Am. Soc. C. E. (by letter).—In the section of this paper devoted to "Pressure Exerted by Cohesive Earth," the writer quite agrees that there are many conditions under which Coulomb's equation does not give even approximate values. The writer would go still further than Professor Terzaghi and suggest that Equation (4) be abandoned entirely. The writer also agrees that, for cohesive soils, the prospects of predicting the lateral pressure successfully by theory are slight. In that case, should one not continue along the line of observations and design methods used and proposed by such men as the late Sir Benjamin Baker,[16] Hon. M. Am. Soc. C. E., and the late J. C. Meem,[7, 17] H. G. Moulton,[8] and E. G. Haines,[7, 8] Members, Am. Soc. C. E.?

The aforementioned engineers all discussed the special case in which the sheeting is driven from the ground downward and not constructed from the bottom upward with backfilling placed after the sheeting is completed. As in the present case, their observations applied to cohesive earth, with lateral pressure exerted against a sheeted trench, typical in the construction of sewers, foundations, and subways. Their observations can be summarized in three statements, as follows:

A. The line of rupture (see Fig. 10) and its relation to the depth of the excavation are independent of the angle of repose. Actually, there is no such thing as an angle of repose in this special case.

B. The line of rupture is independent of the degree of cohesion in the material.

C. The failure of material, in excavating operations ranging from moist sand to hard rock, appears to be a function of unit weight and characterized by an action independent of the nature of the material, provided the depth reached is sufficient to develop pressures in excess of its shearing strength.

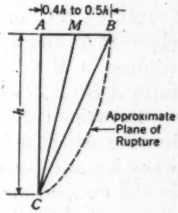

Fig. 10

Three examples may be cited to support the rather broad claim in statement *C*:

Example (*1*).—Sir Benjamin Baker stated, from observations on about 34 miles of deep timbered trenches and tunnels, that on each side "The slope of these fissures was so uniform at the angle of $\frac{1}{2}:1$, measuring from the bottom of the excavation, that the Resident Engineer professed to be able to foretell with certainty where a building or fence wall, standing over the tunnel, would crack most."

Example (*2*).—There are five cases (observed by Mr. Moulton[8]) to support example (2): (*a*) On section 1*A* of route 12 on the double-track tunnel under

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[15] Senior Engr., Federal Works Agency, PWA, Charleston, S. C.

[16] *Minutes of Proceedings*, Inst. C. E., Vol. LXV, p. 140.

[17] "Stresses in Cofferdams and Similar Structures," by the late J. C. Meem, *Civil Engineering*, December, 1934, p. 639.

Flatbush Avenue, Brooklyn, N. Y., a decided break occurred in the ground on either side of the tunnel, at a distance out from the net line of one half the distance from the surface to the top of the tunnel, at the net line; (*b*) gravel banks in placer gold mining operations in Klamath County, Oregon, and Trinity and Shasta counties, California, stood on a 1 vertical : 0.5 horizontal slope; (*c*) placer gold mining operations near Dawson, Yukon, Canada, where the gravel was cemented by ice, also stood on a 1 vertical : 0.5 horizontal slope; (*d*) in stoping operations at Ray, Ariz., Globe, Ariz., and Ely, Nev., the break in the ground occurred on a 1 vertical : 0.5 horizontal slope out from the edge of the stoping; and (*e*) even on such a material as sawdust (in a sawmill at Klamath Falls, Ore., October, 1919) when removed from the bottom of the pile by bulldozers, the sawdust broke at the top on a slope of 1 vertical : 0.5 horizontal. Case (*d*) showed that hard stratified rock, overlain with earth, followed the same law as the gravel.

Example (*3*).—In 1908, Mr. Meem[7, 17] proposed a preliminary wedge theory based on the assumptions built around the angle of repose. At that time he realized that practice did not require rangers to be as heavy as his theory demanded; and by 1934 he had revised his preliminary wedge theory, discarding the angle of repose and building it around the intersection of the plane of rupture and the ground which Messrs. Baker, Moulton, and Haines had placed at one half the depth (provided the depth was great enough to develop pressures in excess of the shearing strength of the material) and which Mr. Meem placed at 40% of the depth.

The practical results of observations *A*, *B*, and *C* were expressed by Mr. Meem as follows: "All the soil in the area *A B C* [Fig. 10] is undoubtedly supported by the face *A C* on one side, and the mean plane of repose [rupture] *B C*, on the other. If it be arbitrarily assumed that one half of this wedge causes pressure against the face *A C*, and that the other half rests on *B C*, then the three problems that must be determined are: 1 * * * the mean plane of repose [rupture] *B C*, or the angle *A C B*; 2 * * * the rational position of the mid-plane *M C*; and 3 * * * the elements and measure of thrust due to the area or volume, *A M C*." Whereas Messrs. Baker, Moulton, and Haines had set the ratio *A B* : *A C* as 0.50 : 1, Mr. Meem set it at 0.4 : 1.

Between 1926 and 1934 the writer had occasion to design timbering and bracing using the foregoing method. Observations on this timbering were that, when a crane (surcharge) was placed next to the cut and was used to set steel in the excavation below, timbering designed on the 1-on-0.4 theory showed distress, but timbering designed on the 1-on-0.5 theory did not. This indicated to the writer that the theories of Messrs. Meem, Baker, Moulton, and Haines were not far from correct.

To illustrate how carefully the foregoing condition must be distinguished from the case of a retaining wall with backfill behind it, a case may be cited in which a sheeted subway cut was intersected by a large runway to provide access for trucks from the bottom of the excavation to the street. It became necessary to close this access, and sheeting and bracing were placed across the opening, in all respects similar to the sheeting and bracing (which was entirely adequate) on each side of the access runway. Cohesive backfill was then dumped behind this sheeting. In the next few weeks, the sheeting, and the bracing opposite the backfill, moved inward 9 in. and had to be reinforced heavily to prevent entire failure.

Another example is that of a subway in which a spur line branched off from the main line in a wide curve. The main line was built first and the load of the sides transferred to the finished subway structure before the branch line was started. The same sheeting and bracing were used on the branch as on the main line and it was constructed toward the main line. When the line of rupture of the branch line intersected the line of rupture of the main line, the bracing showed signs of distress and much heavier timbering had to be substituted in order to bring the branch alongside and into the main line.

The writer would like to emphasize the author's suggestion that the Soil Mechanics and Foundations Division collect data on actual cuts such as those described by Professor Terzaghi and as proposed by Mr. Moulton in 1920[18] and by the writer in 1932.[19]

To recapitulate the writer's views—for cohesive earth, in the special case of sheeting and bracing driven from the ground downward (not the case of a retaining wall with backfill placed behind it):

(1) The author is correct in stating that Equation (4) yields only very approximate values;

(2) The author is also correct in stating that the prospects of being able to predict the lateral pressure successfully by theory are slight; and

(3) The writer proposes that this paper would not be complete without discussion and experiments proving or disproving observations *A*, *B*, and *C*, as described in this discussion.

The writer has confined his discussion to cohesive earth because in actual practice he has dealt with it more often than with absolutely cohesionless material. Acceptable definitions for the following terms would help clarify this matter: Active earth pressure, passive earth pressure, angle of repose, angle made by material pushed off the slope from the top, and the angle made by the line of rupture and the face of a sheeted trench.

In conclusion the writer wishes to thank Professor Terzaghi for his excellent paper.

JACOB FELD,[20] M. AM. SOC. C. E. (by letter).—So much has been written about the "wedge theory," as compared to the usual textbook presentation of the Coulomb theory, that a description of what Coulomb really did publish should be of interest. Most of the later descriptions of the Coulomb theory were apparently copied from either Woltmann's[21] translation (with errors contained therein) or from some later source originating from that translation of the original Coulomb's theory as published by him in 1773.[22] This was published in translation in *Böhms Magazine* in 1779, without change. Coulomb further developed his ideas in his "Théorie des Machines Simples," published in Paris about 1780, a second edition appearing in 1821. In the section on "Sur les Murs de Revêtements et l'Équilibre des Voûtes" (On Retaining Walls and the Balance of Arches), Coulomb covers the subjects of friction, cohesion, rupture of bodies, resistance of brickwork, pressure of earth, area and shape of rupture surfaces, arches without friction or cohesion, and equilibrium of arches, taking into consideration both friction and cohesion.

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[18] "Earth and Rock Pressure," by H. G. Moulton, *Transactions*, Am. Inst. of Mining and Metallurgical Engrs., 1920, p. 20.

[19] *Civil Engineering*, September, 1932, p. 585.

[20] Cons. Engr., New York, N. Y.

[21] "Beitraege zur Hydraulischen Architektur," by R. Woltmann, 1799 (IV Band), 1794 (III Band).

From the foregoing, it is seen that Coulomb merely treated the problem of earth pressure as a part of the general problem of evaluating friction and cohesion as resistances. It must also be kept in mind that Coulomb wrote before the invention of trigonometric symbols or ideas and computed all the ratios as algebraic fractions.

Coulomb lists preliminary propositions in the form of axioms as a basis for his theoretical development. Those which are of immediate interest in this discussion are:

1. If a body is at rest on a horizontal plane, the resultant of the forces acting on it is normal to the plane and passes through the base of the body;
2. These forces can be counterbalanced by a single reaction equal to the resultant and acting opposite to it;
3. Resistance known as friction is proportional to the pressure; and
4. Cohesion is a measure of a resistance which a solid body opposes to disunion into two parts. In homogeneous masses, cohesion is proportional to the area of the section.

Coulomb then develops the value of the necessary resistance of a retaining wall with a vertical face supporting a level fill. The first development is for the assumption of a wedge sliding along a plane, along which there is friction and cohesion. An expression is developed for the active and passive lateral pressure acting normal to the vertical wall in terms of the height of wall, density of the fill, internal friction coefficient (the relationship to the natural slope assumed much later is not mentioned and never occurred in Coulomb's writings), the cohesion of the fill, and the surface width of the wedge of rupture.

In the next section, Coulomb discusses the possibility of a sliding prism along a curved surface, along which both friction and cohesion resistances are acting; but he states that experience (possibly "experiments") shows that the rupture is along a plane surface.

By the application of the theory of maxima and minima, Coulomb then develops an expression for the maximum value of the lateral pressures in terms of the friction and cohesion factors and shows that any cohesion reduces the values; therefore he assumes cohesion to be zero to obtain maximum results. The resulting formula, upon the introduction of the tangent of an angle as the friction coefficient, reduces to the well-known "Coulomb formula" which, for active earth pressure, is

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[22] "Essai sur une Application des Règles de Maximis et Minimis à Quelques Problèmes de Statique Relatives à l'Architecture," by C. A. Coulomb, Vol. VII, Royale Académie des Sciences, Paris, 1776 (1773 volume).

$$P_A = m\,h_w^2 - c\,l\,h_w \quad \text{(5)}$$

in which m and l are coefficients depending on the coefficient of friction, height of wall, and the density of the fill; h_w is the height of wall; and c is the cohesion coefficient. Where cohesion is not disregarded, Equation (5) is used for determining the minimum size of the retaining wall to balance moments about the heel.

By setting P_A equal to zero, Coulomb obtains the formula

$$h_e = \frac{c\,l}{m} \quad \text{(6)}$$

as the height of excavation that will stand vertically unsupported because of cohesion.

The next section takes up the effect of surcharge, as an additional weight to that of the wedge of rupture, taking that part of the surcharge directly above the wedge.

The effect of wall friction is then evaluated by inserting in the general method a reduction of the weight of the wedge equal to the active pressure times the coefficient of friction of the fill against the wall. This coefficient is not assumed the same as the internal friction coefficient.

The final section starts to evaluate the area of the wedge of maximum pressure on an assumed curved surface of rupture, a part of which does not move because of cohesion. The mathematical detail became too involved, and the assumption is inserted: For the "fresh earth case"; that is, cohesion is zero, and a very involved formula is given for the shape of the curved surface of rupture. The basis of the derivation is the computation of maximum pressure additions of vertical earth sections of infinitesimal widths on vertical sections. The variable being the depth of the section,the resulting general equation gives the shape of the curve.

The writer always has been puzzled by the fact that no complete description of what Coulomb really contributed appears in the numerous texts on earth pressure, and it is hoped that its publication herein will correct many false impressions.

The author adds a description of a new thought to the solution of the problem—namely, the manner in which the wall or resistance tends to move in order to cause incipient failure of the fill. It was clear to Coulomb that no resistances would develop without such movement. Since the publication of this paper, there is little to add to the subject of earth pressures against rigid walls which move as a unit. However, the subject becomes infinitely more complicated when the attempt is made to apply the theory to flexible sheathing. In the writer's opinion, the field measurements of lateral pressure in back of sheathing are too indefinite to be of any value. The measurements are made on an indeterminate structure consisting of continuous sheathing supported by a series of wales and struts which were installed without regard to proper accurate location to avoid transfer of reactions. The reactions are a function of pressure intensity and location of the reactions. Slight changes in the bracing as installed change the reactions entirely. This is the explanation for the erratic and inconsistent curves in Fig. 9.

With reference to Fig. 9, there also seems to be a disregard of the reaction taken by the soil from the embedded portion of the sheathing (or soldier beams supporting the sheathing). Even with such omission, the total pressure included in area *A B C D* is 1.12 times the total theoretical pressure. The omission of the reaction at the bottom of the soldier beams explains the high location of the measured pressure, 0.53 h_e to 0.60 h_e, since those values are for the remainder of the pressure only. The reactions on certain struts were measured. It is not clear how these values were translated into pressures on the sheathing. It is hoped that the author will complete the missing steps in his closure.

In recent years so many papers have been published on various phases of soil mechanics which contain so little usable information that this clearly written description of the recent developments of the wedge theory comes as a pleasant surprise and a "landmark" contribution in the literature on lateral earth pressure. The history of the subject was published in 1928 by the writer.[23] The author fills in the copious additions to the subject since then.

M. G. SPANGLER,[24] M. AM. SOC. C. E. (by letter).—The introduction of a logarithmic spiral to define the surface of rupture, instead of the usual straight line, in computations of the lateral earth pressure on a restraining structure by the Coulomb type of theory, is an interesting and ingenious procedure. There is ample justification for assuming the surface of rupture to be similar in shape to this equiangular spiral, since many observations of cracks in the earth behind restraining structures have indicated the essentially vertical orientation of the ruptured surface for considerable depths below the natural ground level and, although actual observations of the lower extremities of the path followed by this surface are nonexistent, there is no reason to doubt that it bends toward and passes through the base of the structure. The author has made a welcome contribution by showing that, even when the surface of rupture deviates from a plane as much as does the logarithmic spiral surface, the error introduced by using the plane of Coulomb's theory does not exceed 6%. This is a comforting revelation.

The dearth of actual measurements of both the distribution and magnitude of lateral earth pressures on the timbering of excavations adds greatly to the interest in that part of the paper which deals with the field measurements of pressures in the struts of a deep subway cut in Berlin. Such measurements, when properly interpreted, are of the utmost value and there is need for many more case studies of this kind.

In the case reported, however, there is a real question as to whether the pressure magnitudes and distributions shown in Fig. 9 are justified by the data obtained in the measuring operations. The steel I-beams which were driven along the outside faces of the cut, and against which the sheathing planks were placed, are shown in Fig. 4 to project downward into the sand and gravel material an appreciable distance below the bottom of the cut. If this illustration is correct, the lower end of these beams must have exerted a sizable horizontal reaction against the sand and gravel below the bottom of the cut, and the sum of the compressions in the four horizontal struts probably represented only a portion of the total lateral pressure acting against the sheathing. These vertical beams, instead of acting as a continuous beam over four supports, as considered by the author, appear in the diagram to have been continuous over five supports, and, since sand and fine gravel would provide a relatively rigid bearing support, it seems likely that this fifth support may have carried an appreciable percentage of the total lateral load. Therefore, the pressure magnitudes and the high center of gravity of the resultant pressures, as shown in Fig. 9, remain uncertain until more definite knowledge of the installation conditions is available.

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[23] "History of the Development of Lateral Earth Pressure Theories," by Jacob Feld, *Proceedings*, Brooklyn Engineers' Club, January, 1928, pp. 61–104.

[24] Research Associate Prof., Civ. Eng., Iowa Eng. Experiment Station, Iowa State Coll., Ames, Iowa.

Donald M. Burmister,[25] Assoc. M. Am. Soc. C. E. (by letter).—In his presentation of the general wedge theory Professor Terzaghi has made a most important contribution to current knowledge of earth pressure and has furnished a more rational basis for the design of retaining structures. The important question now is how can this information be applied practically and in what way should present methods of design be revised? It is hoped that the author will formulate a few principles and criteria for the design of retaining walls, abutments, and similar structures as a guide for design in the future. In the analysis of a given situation a number of questions arise:

(1) What reasonable assumption can be made regarding the angle of internal friction and of wall friction to be used in the computation of earth pressure?

(2) What design criteria and factors of safety should be used to insure permanent stability of a retaining wall?

(3) How do conditions change with time for the different classifications of soils and what minimum requirements for drainage should be specified in the different classifications of soils?

(1) What Reasonable Assumption Can Be Made Regarding the Angle of Internal Friction and of Wall Friction to Be Used in the Computation of Earth Pressure?—The values depend not only on the character of the backfill material and the moisture content, but also on the method of placing and amount of compaction, and whether the fill has been built up toward or away from the wall, or has been built up in level layers. It may be doubtful whether these conditions can be known in sufficient detail in a given case to make a reasonable estimate of the possible limits within which the angles of internal friction and of wall friction may lie. If average values must be used, in the absence of specific information, a tabulation of the possible range of values for the different classifications of soils and for the different methods of backfilling would be very useful, particularly the possible lower limits that should be assumed. If tests are to be made to determine the values, it would be desirable to know what type of test would be considered satisfactory to determine the limits, and how many tests should be made in order to obtain representative values.

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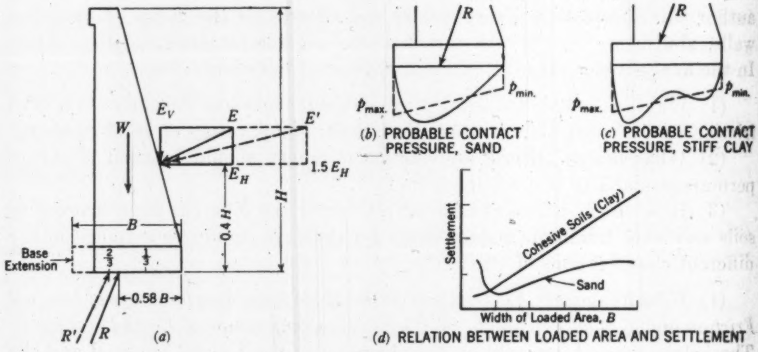
(2) *What Design Criteria and Factors of Safety Should Be Used to Insure Permanent Stability of a Retaining Wall?*—The investigation of stability involves a consideration of a number of important factors. It is obvious that the usual criteria for stability against overturning are inadequate. A consideration of stress-strain phenomena in soils shows that a retaining wall founded on ordinary soils will both settle and tilt a certain amount. The logical design criteria, in the light of this knowledge and of the important influence of wall yield, as demonstrated by Professor Terzaghi,[2,6] are to limit settlement and tilting to such values that the safety of the wall is assured under the most severe conditions to be anticipated, and also that the appearance of the wall is not damaged. The backfill is not an ideal, cohesionless, homogeneous material, the method of placing the fill may have an important influence, and the frictional resistance may not be fully mobilized. It seems logical, therefore, to assume that the center of pressure on the back of the wall should be taken at least at a height of 0.4 H, for the computation of stability, $\Sigma H = 0, \Sigma V = 0, \Sigma M = 0$, and that the base width should be increased so that the resultant pressure on the base computed for the ordinary Coulomb earth pressure would fall between the 0.55 and 0.60 point of the base width, instead of the usual two-thirds point, as shown in Fig. 11(*a*). This has two important effects: First, it makes the settlement of the wall smaller and more uniform; and second, it reduces the maximum toe pressure and thereby reduces tilting. The pressure computed from Coulomb's general equation represents the minimum or ordinary lateral earth pressure condition, which obtains after the wall yields a certain amount, as demonstrated by the author.[6] However, some estimate must be made for the greatest pressure that is likely to occur. This can only be learned, as pointed out by Professor Terzaghi, by accumulating accurate, complete information on retaining wall failures and by carefully analyzing these situations for the possible maximum earth pressure. In the absence of this knowledge it seems reasonable to make the simple assumption that the horizontal component of the earth pressure may conceivably be, for example, 1½ times that computed from Coulomb's formula. This, in effect, means that the frictional forces are not fully mobilized from some cause and that both the angles of internal friction



(*a*)

(*b*) PROBABLE CONTACT PRESSURE, SAND

(*c*) PROBABLE CONTACT PRESSURE, STIFF CLAY

(*d*) RELATION BETWEEN LOADED AREA AND SETTLEMENT

Fig. 11

and of wall friction are low. Under these extreme conditions the resultant must not pass outside of the middle third of the base.

The next factors to be considered, and about which little is really known, are the distribution and intensity of the base pressure for an eccentric loading. The conventional trapezoidal distribution is probably modified in a very important manner, as indicated in Figs. 11(*b*) and 11(*c*). The maximum intensity may possibly be 25% greater than that computed by the conventional method. For a more rational design the load-settlement relations should be determined for the foundation material by means of loading tests, particularly for large retaining walls. The question then arises in the case of a long, continuous footing whether or not allowance should be made for the effect of the width of the foundation on settlement as is done in the design of spread foundations (see Fig. 11(*d*)), particularly in view of the eccentric loading. If presumptive, allowable bearing values must be accepted, it would seem logical to limit the maximum toe pressure by the conventional method to 75% of the allowable. Settlement and tilting records, together with tests by pressure cells placed under the base and on the back of the wall, would yield very valuable information on all of the factors that influence the situation, and would help to explain how conditions change with time from the beginning of the backfilling operation.

The condition for stability against sliding for the horizontal component of the earth pressure, times 1½, is usually satisfied if other conditions are met satisfactorily. The important question here is the lowest value the sliding friction can assume for the different classifications of soils when saturated, and also how much passive pressure can be counted on under the most unfavorable conditions.

(*3*) *How Do Conditions Change with Time for the Different Classifications of Soils and What Minimum Requirements for Drainage Should Be Specified in the Different Classifications of Soils?*—The normal shrinkage or the swelling with saturation of certain soils affects both the magnitude of the earth pressure and the position of the resultant—certain cyclic effects. An important question in railroad and highway embankments is whether the full live load should be considered or only some fraction—for example, 20% in the computation of the earth pressure for retaining walls and abutments. Probably a large proportion of retaining wall failures have occurred during or just after a heavy rainstorm. If proper provisions for drainage have not been installed, hydrostatic pressure on the back of the wall would exert pressures for which few walls are designed. Professor Terzaghi has emphasized the importance of proper drainage and methods to secure it.[26] It is common practice on railroads to backfill abutments for some distance with cinders for drainage purposes, but it is not always possible to use such materials because they are not readily available.

In conclusion the writer wishes to thank Professor Terzaghi for his excellent and timely paper.

Karl Terzaghi,[27] M. Am. Soc. C. E. (by letter).—Mr. Peckworth suggests that the paper would not be complete without discussion and experiments proving or disproving three statements concerning cohesive earth, designated *A*, *B*, and *C*. Since Mr. Peckworth admits himself that there is no such thing as an angle of repose of cohesive earth, a discussion of statement *A* appears to be unnecessary. Statement *B* does not refer to a controversial subject. Every one of the existing theories of earth pressure leads to the conclusion expressed by this statement, and the writer does not know of any observations which are at variance with this conclusion.

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[26] "Effect of the Type of Drainage of Retaining Walls on the Earth Pressure," by Karl Terzaghi, *Proceedings*, International Conference on Soil Mechanics and Foundation Eng., Vol. 1, Paper J-3, Cambridge, Mass., 1936, p. 215.

[27] Dr. Ing., Lecturer, Harvard Univ., Cambridge, Mass.

The observations described by Mr. Peckworth in connection with his statement *C* seem to indicate that the break in the ground behind a vertical bank with a height h always occurs at a distance $\frac{h}{2}$ from the rim of the bank. This statement agrees with the writer's experiences. All the breaks of this category which are known to the writer from personal experience have one important feature in common. All of them started with the formation of a tension crack at a distance $\frac{h}{2}$ from the rim of the bank. Once this crack had developed the bank was permanently weakened and sooner or later it failed along a curved surface of rupture which connected, in some manner, the bottom of the crack with the foot of the bank. This observation led the writer to the tentative conclusion that the tensile stresses on the horizontal surface next to the upper rim of a vertical bank in cohesive earth are a maximum at a distance $\frac{h}{2}$ from the rim of the bank. In order to determine whether this rule is also valid for perfectly elastic materials, the writer made model tests several years ago, in his Vienna (Austria) laboratory, with a vertical bank made of gelatin, and determined the tensile stresses along the upper horizontal surface from the results of strain measurements. The results are shown in Fig. 12. The dotted line represents the deformation of the bank due to its own weight, and the ordinates of the plain curve with reference to the section through the upper horizontal surface represent the horizontal tensile stresses σ_h along this surface. These stresses are a maximum at a distance of about $\frac{h}{2}$ from the rim of the bank. Hence, if a vertical bank of gelatin fails, it fails in the same manner as a bank in cemented sawdust, hard rock, or stiff clay. The mechanical properties of the bank material merely influence, to some extent, the shape of the surface of rupture between the bottom of the tension crack and the foot of the bank.

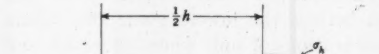

FIG. 12

The preceding conclusions lose their validity for soils without any cohesion, such as clean sands. Fig. 3 shows that the shape of the surface of sliding in a mass of sand behind the vertical sides of a timbered cut is very similar to the surface of rupture in a cohesive material, such as that shown in Fig. 12. For an angle of internal friction, $\phi = 31°$, and an angle of wall friction, $\delta = 26°$, the general wedge theory for cohesionless materials leads even to the conclusion that the surface of sliding in the sand intersects the horizontal surface at a distance equal to one half the depth of the cut. This surface of sliding is practically identical with the surface of rupture, shown in Fig. 12; yet all these similarities are purely accidental and have no significance. For values of ϕ greater than 31° the distance between the upper rim of the cut and the upper edge of the surface should be smaller than $\frac{h}{2}$. Since the angle of internal friction of sands is very seldom smaller than 31°, the width of the top of the sliding wedge adjoining the sheeted sides of cuts in sand should, as a rule, be smaller than $\frac{h}{2}$. The experiences of the writer confirm this conclusion. In some cases the writer found that the measurable settlements of the surface next to cuts in sand did not extend beyond a distance of 0.4 h.

At the end of his discussion Mr. Peckworth requests the definition of several of the terms which are used in connection with earth pressure. The writer suggests the following:

> "Active earth pressure" is the pressure exerted by the earth on a yielding lateral support at the instant when the earth fails by sliding in a downward direction along an inclined surface of sliding.

In sandy soils a very small lateral yield of the support is normally sufficient to reduce the lateral pressure of the earth to a value close to the active earth pressure. Therefore, the designer is usually justified in assuming that the lateral pressure of a sandy soil on a slightly yielding lateral support is approximately equal to the active earth pressure. However, the yield required to produce actual shear failure in the soil can be many times greater than the yield required to approach closely the active earth pressure.

> "Passive earth pressure" is the greatest lateral pressure that can be exerted on a soil by a structure such as an arch bridge abutment or the lower part of a sheet-pile bulkhead.

The mobilization of the passive earth pressure is associated with a lateral compression of the soil. Even in sandy soils the lateral compression required to increase the lateral earth pressure to a value close to the passive earth pressure is very considerable. Hence, if the stability of a structure depends on the lateral resistance of the earth, one should never count on more than one third, or at the most one half, of the computed value of the passive earth pressure.

> The "angle of repose" is equal to the angle between the horizontal and the slope of a heap of soil produced by dumping the soil from some elevation.

For perfectly clean and dry sand or gravel the angle of repose is fairly independent of the height of the heap and the method of dumping, and it is approximately equal to the angle of internal friction of the sand in the loosest state. The angle of repose of moist sand and of cohesive soils depends essentially on the height of the heap and on the method of dumping. Hence, in connection with such soils, the angle of repose has no meaning.

The "angle made by material pushed off the slope from the top" depends on the degree of disintegration prior to the process of pushing and on the weather at the time of pushing. In the rainy season the material may flow off without being pushed. Hence, no definite rules can be established.

The term "angle made by the line of rupture and the face of a sheeted trench" has no definite meaning because this angle increases from a maximum value at the toe of the bank to zero degrees at the upper surface, as shown in Fig. 3. The angle at the toe of the bank depends on both the angle of shearing resistance ϕ and the angle of side friction. If these two values are known, the slope angle of the surface of sliding at the toe of the bank can be determined with close approximation by means of Mohr's diagram.

Referring to the results of the measurements of the pressure in a subway cut in Berlin, illustrated by Figs. 4 to 9, Mr. Feld expressed the opinion that the results of field measurements of lateral pressure in back of sheathing are too indefinite to be of any value because "Slight changes in the bracing as installed change the reactions entirely." It will be shown in the following that the scattering of the measured strut pressures from the average is by no means as important as the appearance of the curves in Fig. 9 would seem to indicate. Nevertheless, no matter how important that scattering may be, every one of the struts must be made strong enough to stand the greatest pressure that may act on it under the most unfavorable field conditions, because the failure of a single strut is likely to cause a failure of the entire bank support. Therefore, every designer has a vital interest in securing information on the greatest possible deviations from the statistical average, and this information can be secured only by pressure measurements in full-sized cuts such as those described in the paper.

Regarding the results of the measurements, Mr. Feld merely commented on the scattering of the empirical pressure distribution from the statistical average. However, from a practical point of view, the designer is chiefly interested in the deviation of the measured values from those theoretical values on which the design is based. Therefore it may be more interesting to compare the measured values with those obtained by theory.

In 1936 the writer proposed,[9] on the basis of theoretical considerations, that the triangular pressure distribution should be replaced by the pressure distribution represented by the trapezoidal area $A\ B\ C\ D$ in Fig. 9. The size of this area is equal to 1.12 times the theoretical pressure P_{an} determined by Equation (1). The factor 1.12 represents a margin for the inevitable scattering of real pressures from the statistical average. In order to compare the errors associated with the traditional and the proposed method of computing the pressures in the struts, assume that the soldier beams are cut immediately below point D in Fig. 13 and hinged at the elevation of the struts Nos. 2 to 4. At point D they are freely supported. The unknown horizontal pressure on the support at D is assumed to be equal to 10% of the sum P_{1-4} of the measured pressures on the struts 1 to 4. Hence, the total lateral pressure on the sheathing is equal to 1.1 P_{1-4} for each profile. The values of the pressures on the individual struts and the data concerning the elevation of the struts with reference to the surface of the ground have been published.[10] For profile 7 these data are given in Fig. 13. At the site of this profile the cut has a depth of 11.5 m (37 ft 9 in.). The measured strut pressures are listed in the figure under the heading "P." The sum P_{1-4} of these pressures is 45.5 tons. (In this profile the pressure in the highest strut has not been measured. In the other profiles it ranged between 6.5 and 8.5 tons. Hence, it is assumed that the pressure in the highest strut is equal to 7.5 tons.) Thus, the total lateral pressure on the sides of the cut adjoining profile 7 is equal to 1.1 times 45.5 = 50 tons. If this pressure acts like a hydrostatic pressure, the triangle *A E D*, Fig. 13, representing a total pressure of 50 tons, is obtained as a pressure area. The strut pressures produced by this load are listed in the figure under the heading "P_c." On the other hand, if the load is assumed distributed over the sides of the cut as indicated by the trapezoidal area *A B C D*, representing a total pressure of 1.12 times 50 = 56 tons, the resulting pressures are as listed in Fig. 13 under the heading "P_t." The three columns of numerical values show at a glance that the errors associated with the traditional method of computation represented by the column "P_c" are intolerable. For the highest strut the measured pressure is more than three times greater than the computed one. On the other hand, the agreement between the measured values and the computed values P_t is very satisfactory.

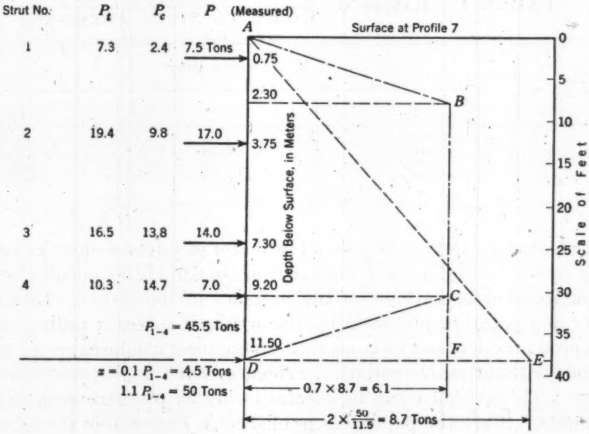
Strut No.
P_t
P_c
P (Measured)
1
7.3
2.4
7.5 Tons
2
19.4
9.8
17.0
3
16.5
13.8
14.0
4
10.3
14.7
7.0
P_{1-4} = 45.5 Tons
z = 0.1 P_{1-4} = 4.5 Tons
1.1 P_{1-4} = 50 Tons
A
B
C
D
E
F
Surface at Profile 7
0.75
2.30
3.75
7.30
9.20
11.50
Depth Below Surface, in Meters
0
5
10
15
20
25
30
35
40
Scale of Feet
0.7 × 8.7 = 6.1
2 × $\frac{50}{11.5}$ = 8.7 Tons

FIG. 13

A similar computation has been made for profiles 4 and 10 which represent the most abnormal profiles in a set of seven. Table 1 contains the ratio between the measured strut pressures P and the values P_t computed by means of the trapezoid method. The table shows, first of all, that the errors associated with the method of computation proposed by the writer are much smaller than those involved in the traditional method of computation (see values of P_a in Fig. 13). Second, it shows that the highest ratio between the measured and calculated values for the strut pressures is considerably smaller than the lowest customary factor of safety. Hence, a low factor of safety seems to be sufficient for taking care of the influence of unequal wedging and other accidental features of the construction operations on the pressure in the struts. However, the method of computation proposed by the writer involves a radical departure from accepted practice, and no conscientious engineer can be expected to change his methods without ample empirical evidence that the proposed modification is justified. This evidence can be obtained only by pressure measurements in full-sized cuts. Since the paper was published in *Proceedings* (October, 1939), such measurements have been made in one cut in Boston, Mass., and in four cuts in Chicago, Ill., with a depth from 30 to 50 ft. Although all these cuts were made in clay, the distribution of the lateral pressure on the sheathing is strikingly similar to that shown in Fig. 9 of the paper, for a cut in clean sand.

TABLE 1.—RATIOS OF $\frac{\text{MEASURED STRUT PRESSURE}}{\text{COMPUTED STRUT PRESSURE}}$

Strut No. (see Fig. 13)	PROFILE NOS.:		
	4	7	10
1	1.19	1.03	1.17
2	0.60	0.88	1.06
3	0.99	0.85	0.91
4	0.77	0.68	0.42

In the last paragraphs of his discussion Mr. Feld commented on the writer's disregard of the reaction taken by the soil from the embedded portion of the soldier beams supporting the sheathing. His objections are shared by Professor Spangler. Both discussers seem to believe that the high position of the centroid of the lateral pressure illustrated by Fig. 9 is chiefly due to the influence of the neglected reaction on the distribution of the lateral pressure. According to this conception, the real distribution of the lateral pressure should be very different from that shown in the figure and much more similar to the hydrostatic distribution.

The writer ignored the lower reaction because he believes that its influence on the distribution of the lateral pressure is too small to be of any practical consequence. The reasoning which led him to this opinion is illustrated by Fig. 14, which is a section through the cut at profile 7. The measurements in the field furnished the values for the pressure on the struts at points 1 to 4. They were equal to 7.5, 17.0, 14.0, and 7.0 metric tons (16,500, 37,600, 31,000, and 15,400 lb), respectively. In order to estimate the corresponding distribution of the lateral pressure, the writer assumed that the soldier beams are hinged at points 2, 3, and 4, and are freely supported at points 1 to 5. Then he determined by trial the outer boundary of a pressure area which satisfies the condition that the pressure is in equilibrium with the measured reactions at the points 1 to 4. The result of this operation is shown in curve 7, Fig. 14.

In reality the soldier beams are continuous and they are bent as indicated by the broken line in Fig. 14. In order to produce a deflection of that type one must add to the strut reactions P_1 to P_4 a soil reaction X. At the same time one must shift the curved boundary of the pressure area from its original position 7 into a position 7′ in such a manner that the pressure represented by the shaded area shown in Fig. 14 is equal to the reaction X. The pressure curve 7′ thus obtained resembles the theoretical pressure curves computed by the writer[9] in 1936. The shaded area shown in Fig. 14 represents 10% of the measured pressures P_1 to P_4 on the struts 1 to 4. The soldier beams in the cut in Berlin consisted of I-beams, 12 in. wide, which were driven to a depth of not more than 5.5 ft below the bottom of the cut. Therefore, the writer believes that the effect of the neglected soil reaction X on the distribution of the lateral pressure should be appreciably smaller than that shown in Fig. 14.

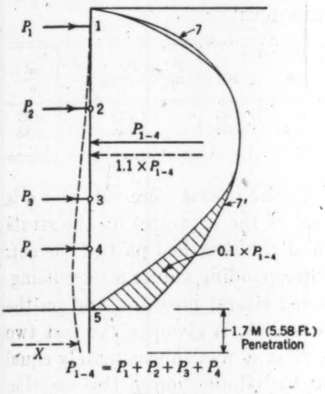

FIG. 14

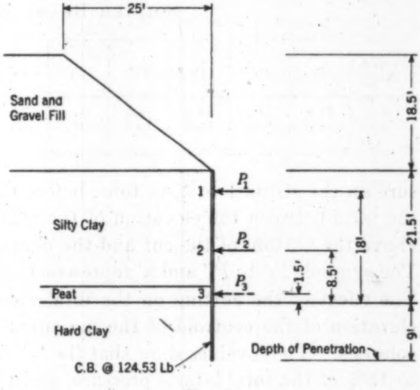

FIG. 15

In order to prove or to disprove this opinion, it would be necessary to cut the soldier beams at point 5 and to measure the effect of this operation on the strut pressures. No such operation has been performed in Berlin. However, quite recently, A. Casagrande, Assoc. M. Am. Soc. C. E., has made a similar test on a cut in Boston, because the conditions were such that the soil reaction X was likely to be exceptionally important. Fig. 15 is a section through the cut. The soldier beams were much heavier than those used in Berlin and, although the depth of the sheeted part of the cut was only 21.5 ft, the beams were driven to a depth of 9 ft below grade into hard clay. To a depth of 18.5 ft the cut was made with sloping sides, as shown. The remainder of the cut was made with vertical sides. The lagging consisted of horizontal boards resting on the inner side of the flanges of the soldier beams. At points 1 and 2, Fig. 15, the soldier beams were supported by struts. At point 3 a strut was loosely fitted to the soldier beam without carrying any pressure. After the pressures P_1 and P_2 on the struts 1 and 2 had been measured, the vertical soldier beam was cut immediately below point 3 and the pressure on the struts was measured once more. In Table 2 the symbols P_1 to P_3 represent the pressure on the struts 1 to 3, in tons, before the soldier beams were cut, and n is the ratio between the elevation of the centroid of the pressures on the struts above the bottom of the cut and the depth h of the sheathed part of the cut. The symbols P_1' to P_3' and n' represent the corresponding values after cutting. The effect of the cutting on the total measured lateral pressure, and on the elevation of the centroid of the measured pressures, is given in the last two columns. These values show that the soil reaction X was approximately equal to 15% of the total lateral pressure, and that the elimination of this reaction lowered the centroid of the pressure by about 15%. In order to compare the conditions for the end support of the soldier beams in the cuts in Berlin and the cuts in Boston, respectively, the following data may be studied:

TABLE 2.—PRESSURE ON STRUTS 1 TO 3 BEFORE AND AFTER SOLDIER BEAMS WERE CUT

Profile No.	P_1	P_1'	P_2	P_2'	P_3	P_3'	n	n'	$\frac{P_1' + P_2' + P_3'}{P_1 + P_2 + P_3}$	$\frac{n'}{n}$
1	11.8	12.2	10.8	9.9	0	4.6	0.76	0.65	1.154	0.85
2	13.0	12.5	15.3	15.4	0	5.1	0.73	0.62	1.145	0.85

Item	Description	Berlin	Boston
1	Total depth of penetration, in feet............	5.5	9.0
2	Ratio, $\frac{\text{height of sheathed sides}}{\text{total depth of penetration}}$............	7.2	2.4
3	Weight of soldier beams, in pounds per foot....	50.0	53.0
4	Width of soldier beams, in inches.............	12.0	12.0
5	Width of flanges, in inches..................	6.0	12.0
6	Area of contact between soil and the outer face of the flange in the buried part of the soldier beam	2.8	9.0

These values show that the lower ends of the soldier beams in Berlin were very much less confined than those in Boston, and they were much more flexible. In Boston the soil reaction X was about equal to 15% of the total lateral pressure. Hence, the writer feels justified in assuming that the influence of the neglected soil reaction X in Berlin on the intensity and the distribution of the lateral earth pressure was much less important than the influence indicated by the shaded area in Fig. 14. In order to eliminate, entirely, the error due to disregarding the soil reaction X, future measurements in cuts with soldier beams should be combined with measurements of the effect of cutting the soldier beams on the pressure in the struts. The author dares to predict, however, that the influence of the cutting on the pressures in the struts will usually be very small. This forecast agrees not only with the preceding analysis but also with the following field observation: In one of the cuts in Chicago the lower ends of the soldier beams were located above the bottom of the cut.

Nevertheless, the centroid of the measured strut pressures was close to the midheight of the sides of the cut, and the value of the ratio between the elevation of the centroid and the depth of the cut is located within the range of scattering of the corresponding value for the other cuts, with soldier beams whose lower ends were driven 5 ft into the ground.

The method of computation represented by the trapezoidal area *A B C D* in Fig. 13 can be used if the system of bracing includes soldier beams with their lower ends embedded in the ground. If no such soldier beams are used, it is advisable to replace the trapezoidal area *A B C D* in Fig. 13 by the area *A B F D*. The area *C D F* represents a substitute for the shaded area in Fig. 14.

In his discussion Professor Burmister presents interesting suggestions regarding the methods of determining the angle of shearing resistance, the distribution of the soil reactions over the base of retaining walls, and the effect of drainage on the earth pressure. However, all these topics are beyond the scope of the paper.

In conclusion the writer wishes to express his gratitude to the discussers for their stimulating comments.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 2100

THE RÔLE OF THE ENGINEER IN AIR SANITATION

A SYMPOSIUM

PAGE

A General Survey.
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WITH DISCUSSION BY MESSRS. THEODORE HATCH, DONALD FRANCIS GRIFFIN, GORDON M. FAIR, H. G. DYKTOR, GILBERT H. DUNSTAN, CHARLES LUNDY POOL, AND J. J. BLOOMFIELD.

NOTE.—Presented at the meeting of the Sanitary Engineering Division, New York, N. Y., January 20, 1938, and published in November, 1939, *Proceedings*.

A GENERAL SURVEY

By Earle B. Phelps, Esq.[1]

Synopsis

Air sanitation may be defined as the control of man's atmospheric environment, for the purpose of meeting the essential physiological requirements, and of avoiding abnormal and harmful conditions. Such a definition automatically places air sanitation within the field of Public Health Engineering, its unique feature being that the fundamental data are of the biological sciences, and its primary objective the preservation of health through environmental sanitation.

The ventilating engineer has long shown his ability and expressed his willingness to modify air to meet any specific requirements, just as the structural engineer can readily design a sewage disposal plant to perform any required physical duty. In each case, however, the requirements must be established through biological investigation, and their ultimate expression may not be in common engineering terms. Nowhere is this unique feature of Public Health Engineering better illustrated than in the field of air sanitation, in which one is forced, at the very outset of any general discussion, to depart entirely from engineering as such, and to turn one's attention to the essentials of a sanitary air environment. It is this field of inquiry—more properly termed "air hygiene"—that the writer attempts to explore herein, with no apology for its obvious inadequacy as to detail or incompleteness as to scope, and with only such occasional reference to the complementary engineering or control measures as seem to be indicated.

The Out-of-Doors Atmosphere

Man's relation to his atmospheric environment begins with the outdoor atmosphere—the much discussed weather. Despite Mark Twain, people are really doing something about the weather. By means of clothing, umbrellas, shut-in cars, and other sheltering devices they do maintain, even in their travels, a kind of personal weather-environment; and, when they confine the weather within walls, they do with it as they will. However, in a general sense, when people go outdoors they still meet "Nature in the raw." The only significant and successful modification of the open air is its pollution by smoke, dust, and gases. The extent and harmful result of such pollution has long been a matter of investigation, discussion, and attempted control. The direct health implications are many, as to possibilities, but few as to definite evidence. Occasionally, however, a successful large-scale experiment is performed.

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An example of gross atmospheric pollution and of widespread damage to life is the famous "Fog Disaster" of the Meuse Valley in Belgium in 1930. Over a period of three days there were several thousand acute pulmonary attacks resulting in about 60 deaths. Cattle, birds, and even rats were killed. Various explanations, mostly designed to exonerate the industries, have been suggested. Some evidence indicates fluorin poisoning resulting from the use of fluorid in the steel and certain other industries.

Over a considerable area in Staten Island, N. Y., investigation has disclosed a sufficient concentration of sulfur dioxide in the air to cause much throat irritation and coughing, and definite damage to garden crops. The worst conditions are found during humid and foggy weather, with light winds from the direction of the industrial plants several miles away in New Jersey.

A comprehensive survey of conditions in New York City has been reported by Sol Pincus, Assoc. M. Am. Soc. C. E., and A. C. Stern.[1] Briefly, the survey shows the extent of air pollution as measured by total soot-fall, classified into soluble and insoluble matter. The former is analyzed further for sulfate, chloride ammonia, and ash, and the latter for tar, combustible matter, and ash. Control stations were established in outlying areas. The air was examined for various gases, sulfur dioxide (SO_2), ammonia (NH_3), and carbon dioxide (CO_2), and for dust and pollen.

The extensive studies of the Mellon Institute at Pittsburgh, Pa., supported by observations from numerous other places, show a marked interference with normal solar radiation, especially in the short-wave end of the visible spectrum and in the ultra-violet. Sunlight is the recognized dominant etiological factor in rickets. The frequency of occurrence and the severity of cases is maximum in the industrial cities, and minimum in the rural areas. Furthermore, normal sunshine possesses marked germicidal properties which are maximum in those rays that are most reduced by smoke. In so far as this process of natural, sunshine disinfection is of value in reducing the prevalence of infectious diseases, smokiness is harmful.

The breathing of dusty air, more or less continuously, may result in interference with normal respiratory functions, in a merely mechanical way, or by setting up specific reactions. The Pittsburgh investigations have shown a reasonably consistent correlation between smokiness and pneumonia in various parts of the city. The investigators attribute the result to mechanical interference with lymphatic drainage—whereby lungs ladened with soot are poorly equipped to oppose infection—rather than to interference with the normal germicidal properties of sunlight.

Finally, an increasing knowledge of hay fever and other allergic manifestations have led to a considerable study of pollens in the air and to a definite program of eradication. An excellent example of this is the work done by the Department of Health of New York City. Regular air examinations were made at many stations over the city areas, and the pollens were counted and identified as to botanical genus. These data were used to direct the work against ragweed, which for many years has been a routine activity in this department, as in many other health departments.

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[1] "A Study of Air Pollution in New York City," by Sol Pincus and A. C. Stern, *American Journal of Public Health*, 27, April, 1937, p. 321.

It must be freely admitted, however, that the economic argument against air pollution thus far has been more powerful than the hygienic argument in stimulating the clean-air movement. Here the issue is clear-cut. Abatement in many cases results in ultimate economy to the offending power plant, as well as material advantage to the households, shopkeepers, and average citizens.

The Indoor Air

Within doors, air sanitation is much more clearly defined. It is convenient to separate industrial conditions from those due merely to human occupancy. For the most part, the latter are uniform and predictable. They form the basis of the modern arts of heating, ventilation, and air conditioning. Industrial conditions cover the widest possible range, in both the quantitative and the qualitative sense.

Any effective control of the indoor atmospheric environment must first satisfy the requirements of human physiology. It has long been known, for example (and may be recorded as the first physiological principle), that the harmful effects of so-called "vitiated" air are due to its physical rather than to its chemical properties. Confined air first fails to function as a suitable environment by its reduced capacity to remove the waste heat from the human body. Much later, if at all, it fails in its capacity to support respiration.

From an engineering viewpoint, the problem of the removal of excess heat from a given body is an exceedingly simple one; to the physiologist, dealing with the human body, it is most complex. After years of investigation, during which many "blind alleys" have been explored and some most promising paths have had to be retraced, the situation at which physiologists have now arrived is essentially as follows:

Heat leaves the body by various avenues—by conduction, convection, and evaporation of moisture, and, from the skin, by radiation. Thermal equilibrium, in a physiological sense, is much more than a mere numerical balance between heat production and heat removal. It is a most delicate balance among the various avenues of heat loss, intimately and automatically connected with each other and with the thermal gradients of the body—the whole being responsible for that sense of well-being known as comfort, as well as for certain reactions and conditions definitely associated with resistance to infection.

Prof. C. E. A. Winslow and his colleagues at Yale University, New Haven, Conn., have stressed in their work[3] the idea of partitional calorimetry, the calorimetry of the body being divided into its various avenues of heat loss and gain. One of the "blind alleys" of the past, exploration of which has impeded advance, has been the concept of equivalent conditions. Under this concept, an excessive rate of heat loss by one channel (evaporation, for example) can be balanced exactly by decreasing the losses in other channels. Out of this concept have come such terms as "effective temperature" in which the total of all cooling effects is supposedly summed up in one item. Unfortunately for any such simplification, merely physical equivalence is not physiological equivalence. Comfort and health demand a balance among the various manners of heat loss in addition to an over-all balance between production and total loss. As the writer has stated this proposition in another place,[4] "An arctic explorer, in his furs, and a swimmer in the surf of Atlantic City may both be disposing of the same number of calories per hour, but one would hardly class the two environments as equivalent."

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[3] "A New Method of Partitional Calorimetry," by C. E. A. Winslow, L. P. Herrington, and A. P. Gagge, *American Journal of Physiology*, 116, August, 1936, p. 641. See also series of papers in this Journal and in the *American Journal of Hygiene*, Vol. 26, 1937.

Just what does constitute the most favorable indoor atmosphere as regards its purely physical nature? Temperature, humidity, and movement are recognized as among the primary variable properties associated with thermal equilibrium in the partitional sense. To these one must add an important feature not at all related to the physical properties of the air itself—namely, the temperature of the enclosing walls. L. B. Aldrich has shown[5] by direct measurement that radiation to surrounding walls, under normal indoor conditions, may account for nearly one half of the body's heat loss. Obviously, various adjustments of these four environmental conditions will result in corresponding adjustment of the internal conditions and a readjustment of the manner of heat loss. It is this finer adjustment rather than an over-all balance that forms the basis of present-day research and that promises to clarify the long studied, but elusive, question stated at the beginning of this paragraph.

The problem has been approached largely from the empirical side. The rather crude but suggestive data of the older physiologists have but slowly filtered into the practice of ventilation. Old errors have better withstood the process of infiltration than has the truth; hence the carbon-dioxide basis of ventilation standards, scarcely discarded to this day. In the "cut-and-try" method of advance, the criterion of "comfort" has been generally used. In some few instances, notably in recent studies of air conditioning, the more fundamental criteria of incidence of respiratory disease, working efficiency, absenteeism, labor turnover, etc., have been reported. The distinction is important. Comfort is largely a matter of adaptation. Undoubtedly, residents in the United States are adapted to an unhygienic indoor temperature, as foreign visitors never fail to point out. On the other hand, the comfort test has indicated certain unknown factors not correlated with the physical conditions ordinarily measured. On the whole the empirical approach has definitely advanced the art.

The experimental approach has been less aggressive, more spasmodic, but of definite value. It has at least exposed certain of the fetishes—the carbon-dioxide and 30-cu-ft-per-min fallacies, for example—and like any good scientific work has corrected some of its own earlier errors, such as the relation of humidity to cooling during the winter heating seasons. It has drawn attention to an important principle—that the external conditions must be adapted to the internal. Different air conditions are necessary to meet the variations of age, and of occupational activity. Finally, those working on the experimental approach are undertaking a detailed study of the physiological reactions involved in body cooling, under all combinations of the environmental variables, and are doing this for the first time as a contribution to air hygiene research rather than to general physiology.

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[4] "Environment in Relation to Public Health," by Earle B. Phelps, *The DeLamar Lectures*, 1926–27, p. 192, Baltimore, 1927.

[5] "A Study of Body Radiation," by L. B. Aldrich, Smithsonian Miscellaneous Collections, 81, No. 6, 1928.

AIR BACTERIOLOGY

An entirely distinct phase of air sanitation—that of air-borne infection—has been brought to the attention of students, with new emphasis, by the work of W. F. Wells[6] who has developed an improved method for bacteriological air examination, and has investigated the bacterial content of air in many places. He revives, with new evidence, the discarded views of earlier days and suggests that air bacteriology must begin, as did water bacteriology, with quantitative and qualitative examinations, and the search for significant correlations with total numbers of bacteria or with specific indicators of harmful contamination.

Summarizing their work W. F. Wells and M. W. Wells[7] show that mouth spray, in part, settles to the ground as droplets; but that, in larger part, it evaporates, leaving nuclei with their contained bacteria capable of floating in the air like tobacco smoke. Various organisms, characteristics of the human respiratory tract (including certain pathogenes and viruses), are capable of surviving, when thus suspended in air, for periods of from an hour to at least two days. Organisms sprayed into the air of the basement of a three-story building were later recovered throughout the corridors of the building. Influenza virus sprayed into an experimental chamber remained suspended and viable for at least an hour. The number of alpha streptococci found in the air of various places corresponds to the density of occupation. The possibilities of aerial infection over a considerable area, as between hospital wards, and over a length of time, is thus manifest. Sterilization of air by ultra-violet light is shown to be effective, and its feasibility upon a working scale has been demonstrated.

Leon Buchbinder, M. Soloway, and M. Solotorowsky[8] have reported the results of examinations of more than 7,000 samples of air from schools, subways, theaters, etc., in New York, N. Y. They find a close correlation between the numbers of alpha hemolytic streptococci and conditions of occupancy and of ventilation. The numbers were higher in schools than in the more congested but better ventilated subway cars; they were higher in unventilated, than in well ventilated, theaters; and they were least in the open parks.

Sanitarians are beginning, therefore, to accumulate data for a bacterial measure of air pollution, and of ventilation and air cleansing efficiency. Although it is too soon to attempt any generalization it may be said fairly that airs may now be compared as to their relative contamination by organisms of the human respiratory tract, and that engineers are now in a position to discuss methods of ventilation and of air cleansing from a bacteriological viewpoint, and thus to supplement the physiological findings as to optimum conditions for thermal equilibrium, with bacteriological findings as to permissible limits of contamination.

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[6] "Apparatus for the Study of the Bacterial Behavior of Air," by W. F. Wells, *American Journal of Public Health*, 23, January, 1934, p. 58.

[7] "Air-Borne Infection," by W. F. Wells and M. W. Wells, *Journal*, Am. Medical Assoc., 107, November 12, November 28, 1936, pp. 1698, 1805.

[8] "Alpha-Hemolytic Streptococci of Air," by Leon Buchbinder, M. Soloway, and M. Solotorowsky, *American Journal of Public Health* ,28, January, 1938, p. 61.

Industrial Conditions

The manifold problems of air sanitation in industry are so diverse as to constitute a distinct field of study which one cannot hope to explore in a brief survey such as this. Generally, they are of the same nature as the problems of normal air conditioning but upon an exaggerated scale, dealing with extremes of heat, humidity, cold, dustiness, and chemical pollution. Many of these conditions are inherent and unavoidable, as in a steel mill or a refrigerating plant. Others, such as the hot, humid air of deep mines, may be remedied only at great cost. The airs of working places, polluted with dusts, fumes, vapors, and gases, are more readily controlled, and a brief glance at some of the types of problem involved may be of interest.

Passing over these industrial conditions in which dust is objectionable primarily because of its concentration and mechanical effects upon the respiratory tract, two typical dusts or fumes of specific injurious nature may be noted. In mining, quarrying, and rock excavation, the breathing of the dust which results from drilling quartz, or certain silicate rock, leads in time to a pathological condition of the lung tissue, known as silicosis. Pulmonary tuberculosis later develops in many of these cases. Adequate ventilation, wet-drilling methods, or protective masks are used to offset this danger.

Finely divided metallic oxides, zinc, lead, etc., arise from the hot or molten metals in many metallurgic processes. Breathed in sufficient concentration they give rise to a condition known as metallic fever. In addition, a specific toxic effect of the metal itself may also be experienced, as in lead poisoning.

Toxic vapors and gases arising from innumerable chemical operations each presents a separate problem, including a physiological study of toxicity and permissible limits in the air. The former extensive use of benzol, for example, in spray painting is typical. Rather widespread prevalence of benzol poisoning resulted before the seriousness of the situation was realized and other solvents substituted. As an indication of the extent of this field, the toxicology and pharmacologic reactions of 32 different volatile substances in the single chemical group of halogenated hydrocarbons used in industry have been studied. Ventilation based upon good engineering practice is the general preventive measure. Before the problem is ready for the ventilation engineer, however, two sets of data are required. Specific gravity, temperature, and other physical data, together with a knowledge of the industrial process itself, determine the best type of ventilating procedure and the proper location of hoods, duct inlets, etc. In addition, ventilation practice must be based upon adequate physiologic knowledge of human tolerance and permissible concentrations.

Vehicular tunnels, such as the Holland Tube between New York and New Jersey, present a unique situation, approximating industrial conditions. In this case a study was made of human tolerance to carbon monoxide, upon a concentration, exposure-hour basis, and of the average rate of production of the gas by various types of motor vehicles. These determined the necessary rate of air supply and exhaust. All this, be it noted, preceded the study of the actual ventilation problem itself. The carbon monoxide contamination of the general atmosphere by motor vehicles, and especially of the air within heavy buses by their own exhaust gases, was studied by Messrs. Pincus and Stern,[2] and has received some study elsewhere.

From the foregoing brief survey it is evident that a field of knowledge and of administrative control is outlined herein which is much broader than that customarily included within engineering. It will readily be discerned, however, that this is not a novel situation. The design and operation of a water, or a sewage, treatment plant or of a milk handling and pasteurizing plant, the control of mosquitoes, and many similar projects engage engineering ability and utilize engineering knowledge to accomplish certain purposes that are expressed only in terms of biology and chemistry. It is believed that the public health engineer must be prepared, with the necessary collaboration from the associated sciences, to assume ultimate responsibility and control in all such cases. The purpose of this paper is to indicate the scope and basis of air sanitation—that is, the control of the atmospheric environment—regarded as an engineering project. The field is already occupied in several of its parts by various engineering specialists. A second purpose of this paper is to indicate the essential unity of this field, the interdependence of its parts, and its ultimate dependence as a whole upon the biological sciences.

SUMMARY

Air sanitation, the control of man's atmospheric environment, is a field in which the public health engineer is called upon for guidance and in which his responsibilities are becoming increasingly evident with the growth of knowledge of the relation between that environment and human health and comfort. The successful solution of its manifold problems demands a somewhat intimate knowledge of those relations, which in turn are based upon the biological sciences, physiology, bacteriology, bio-physics, and bio-chemistry.

Pollution of the general atmosphere results from the discharge of smoke, gases, and fumes, and from industrial or household stacks. It reduces or eliminates the useful ultra-violet radiation, as well as the actual sunshine; it interferes with respiration; it leads to annoyance, minor ills, acute sickness, and even death; and it causes damage to crops, to structures, and to goods.

The problems of heating, ventilation, and air conditioning deal with the atmosphere indoors, subject, in the simplest case, merely to the effects of human occupancy. The determination of what constitutes the ideal indoor air for health and comfort is still undergoing study, but with a more concerted and direct attack by physiologists and with promise of a satisfactory outcome. The reactions of a human subject are not the mere physical reactions of an inanimate body, and the finer mechanism by which man maintains his thermal equilibrium is highly complex and interrelated. No single formula can express these relations and no gross integration of external conditions can meet them.

In modern days, the older theories of air-borne infection have been entirely rejected or held unlikely of practical usefulness. This has been due to inherent weaknesses of the theories themselves (most of which antedate modern bacteriology), and to the identification of many other avenues of infection, such as food, drink, insects, and direct contact. Newer studies in air bacteriology have served to show the total or partial inadequacy of these avenues (in the case of the respiratory infections in particular) and the need to reexamine available data on air-borne infection. These studies are also providing a basis for the practical evaluation of methods of ventilation and air cleansing.

Industrial hygiene deals largely with the healthfulness of environmental air conditions in industrial plants. The field is a broad one covering every phase of ordinary heating, ventilating, and air conditioning, but in an exaggerated degree, and in addition many new problems of specific nature. The ill effects of the inhalation of quantities of any inert dust, the specific effects of certain dusts such as silica or hot metal fumes, the toxic effects of many volatile chemical substances such as benzol, and the ill effects of extremes of heat, cold and moisture, and of sudden changes to and from any of these, are some of the important problems involved.

In his study of methods of air sanitation, the engineer must know the possibilities of good engineering practice and combine with this knowledge a knowledge of the physical, chemical, bacteriologic, physiologic, and toxicologic properties of the special types of impurities with which he has to deal. This constitutes indeed a much broadened definition of engineering and will doubtless require, as in the case of water supplies and food sanitation, cooperative effort among specialists; but, it is to the public health engineer that the future must look for guidance and responsible control.

TYPICAL PROBLEM IN INDUSTRIAL SANITATION

By J. J. Bloomfield,[9] Esq.

Synopsis

Industrial hygiene has been defined as the science of the preservation of the health of the industrial worker. Although it has been known from ancient times that ill health and premature death were often associated with the nature of man's livelihood, it is only in recent years that studies have clearly indicated that the health of workers engaged in industry may be affected by the materials and processes used. For these reasons one of the important phases of industrial hygiene is the study of the relationship between the working environment and its effect on health and methods of control. This phase of industrial hygiene may aptly be termed industrial sanitation, and is a function coming within the province of the engineer.

It is the purpose of this paper to discuss the rôle of the engineer in industrial sanitation studies, and to indicate the trends in this field of public health.

Classification of Health Hazards

Before discussing the various problems confronting the engineer in industrial sanitation, it may be well to define the present conception of industrial health hazards. There are several classifications of health hazards in industry, and the following is only one version. Industrial health hazards may be divided into three types: Chemical, biological, and physical. Under chemical hazards is the important group of poisons, of which there are a great number and variety. L. I. Dublin and R. J. Vane[10] list about 94 groups of industrial poisons in the United States, associated with approximately 2,500 occupational exposures. Health hazards such as poisons may again be classified according to their physiological and pathological effects on the human system. Biological health hazards include the infections, such as anthrax, tuberculosis, typhoid fever, respiratory, and venereal diseases. The physical health hazards are extremely important in industry; and this classification embraces those dusts which may cause fibrosis of the lungs, accidents caused by machinery and other environmental conditions, excessive humidity, heat and cold, and abnormal atmospheric pressure.

From the foregoing suggested classification of industrial health hazards, it should be evident that the practice of industrial hygiene is primarily within the sphere of two types of workers, the physician and the engineer. It is the physician's task to recognize the existence of those diseases associated with the working environment, while the engineer's function is to study the working conditions that may be detrimental to health, and, by precise, quantitative measurements, to determine the extent of the hazard. Once the nature and degree of the hazard have been demonstrated, it is also the engineer's problem to evolve methods for controlling or minimizing the hazardous condition, and finally to study the effectiveness of these measures.

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[9] Sanitary Engr., U. S. Public Health Service, Washington, D. C.

[10] "Occupation Hazards and Diagnostic Signs," by L. I. Dublin and R. J. Vane, *Bulletin No. 582*, Bureau of Labor Statistics.

ENGINEERING ASPECTS OF INDUSTRIAL SANITATION

The magnitude of the problem confronting the engineer in industrial sanitation is as large as it is varied. Industrial sanitation surveys and the various engineering methods followed in controlling industrial health hazards have been discussed fully in previous publications.[11, 12] In this paper the rôle of the engineer in industrial hygiene will be demonstrated from the viewpoint of a mass attack on one particular industry. It is felt that the methodology described for a study of the hazards in this industry may be applied with slight modifications to engineering studies of other industrial problems.

In brief, engineering studies of the working environment involve the determination of the occupational exposure to such air-borne materials as dusts, vapors, fumes, mists and gases, and to substances which may cause corrosive burns or dermatoses. Industrial sanitation also includes studies of the physiology and mechanics of ventilation, illumination, exposure to extreme conditions of temperature, humidity, noise, studies of fatigue, general plant sanitation—in fact, all phases of the environment which may have some bearing on health. In addition, it is evident that in order to have a broad perspective of the entire problem, it is essential that the engineer be at least familiar with the toxicology of many of the materials used in industry and have some knowledge of industrial legislation and labor economics in general.

In the past, those responsible for industrial hygiene administration have been primarily concentrating their efforts on individual plant or workroom studies. It is evident that such procedure, when practiced in some industrial centers or states, would not result in a rapid material improvement in the environmental conditions for a large portion of the working population. Students of the problem of industrial sanitation now realize that more rapid strides may be made by a study of representative plants of an industry, the results of which may then be applied to the entire industry. A description of one of the studies conducted by the U. S. Public Health Service should serve to demonstrate some of the activities involved in industrial sanitation. The example that follows concerns primarily the application of engineering studies to the hatters' fur cutting industry and exemplifies the approach to the problem of industrial sanitation.

METHODS AND INSTRUMENTS USED IN THE STUDY

At the time of this study thirty-six plants in the United States were engaged in the preparation of hatters' fur, employing approximately 2,000 persons.

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[11] "Preliminary Surveys of the Industrial Environment," by J. J. Bloomfield, *Public Health Reports*, Vol. 48, No. 44, November 3, 1933.

[12] "Engineering Control of Occupational Diseases," by J. J. Bloomfield, *Public Health Reports*, Vol. 51, No. 21, May 22, 1936 (Reprint 1749).

Since it was not feasible to examine every worker in the industry, it was necessary to make a selection of plants. This was accomplished by a preliminary survey, which consisted of collecting such data as industrial welfare facilities afforded the workers, the general sanitation of the plant, and details concerning the processes and occupations involved in the preparation of hatters' fur. Certain other information was obtained, such as facilities for conducting physical examinations, and cooperation of employers and employees in the conduct of the study, all of which influenced the selection of the plants. In this manner it was possible to choose five plants proportionately representative of modern and old practice, as well as those which had, on visual inspection, good, fair, or poor working conditions. In addition, this preliminary survey served to give a fairly good "picture" of the entire industry.

Further preparatory steps prior to actual quantitation of the working environment consisted of making sanitary and occupational surveys of the various workrooms in the five plants. Such preliminary surveys serve as a guide for the more detailed studies which follow. The sanitary survey, for example, lists the various facilities afforded the employee in the working environment, whereas the occupational survey permits one to determine the activities involved and the particular hazards associated with each occupation. Survey forms similar to those described elsewhere[11] served as a basis for recording the essential data on the sanitary and occupational phases of this study.

The preliminary survey showed that the major occupational hazard in need of detailed investigation was the exposure to mercury vapor and dust. Certain occupations involving an exposure to fur dust made it advisable to obtain a particle count of this exposure also. For the determination of mercury vapor, use was made of a special selenium sulfide detecter[13] because of its portability, ruggedness, simplicity of operation, and the rapidity of obtaining immediate results. In the present study, mercury dust was excluded from the detecter by placing a single-thickness "paper thimble" at the point where air enters the instrument. The dust-free air then passed into the device, was preheated to a temperature of 70° C, and then allowed to come in contact with a sensitized paper. The method is based on the reaction between active selenium sulfide coated on the paper, and the vapor. Whenever mercury vapor comes in contact with the paper, it is blackened, the degree of blackness being an indication of time of exposure, concentration of mercury, and other factors that can be controlled definitely. The quantity of mercury vapor in the air can be determined by comparing the sample obtained with standards which accompany the instrument.

In order to determine the quantity of mercury dust in the air, samples were obtained with the impinger apparatus.[14] Since it was found in preliminary observations that the mercury coated fur was of a greasy texture and hence difficult to wet, it was necessary to use a 25% alcohol and water mixture as a collecting medium. Samples thus obtained were shipped to the central laboratory at Washington, D.C., where they were analyzed.[15]

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[13] "Selenium Sulfide—A New Detector for Mercury Vapor," by B. W. Nordlander, *Industrial and Engineering Chemistry*, Vol. 19, No. 4, April, 1927.

[14] "Determination and Control of Industrial Dust," by J. J. Bloomfield and J. M. DallaValle, *Public Health Bulletin No 217*.

[15] "The Determination of Mercury in Carroted Fur," by F. H. Goldman, *Public Health Reports*, February 19, 1937 (Reprint 1804).

A Konimeter was used for the determination of the quantity of dust to which the workers were exposed.[16] In the control studies which followed, certain ventilation readings were made on the exhaust systems and these were obtained with the pitot tube.

PREPARATION OF HATTERS' FUR

The processes in the manufacture of hatters' fur may be considered roughly in three parts: (1) The preparation of the pelt; (2) the carroting and drying of the fur; and (3) the cutting or shearing of the hair from the hide. These constitute the basic steps in all hatters' fur establishments. Such differences as may be found among various plants pertain chiefly to the degree to which some departments have been mechanized.

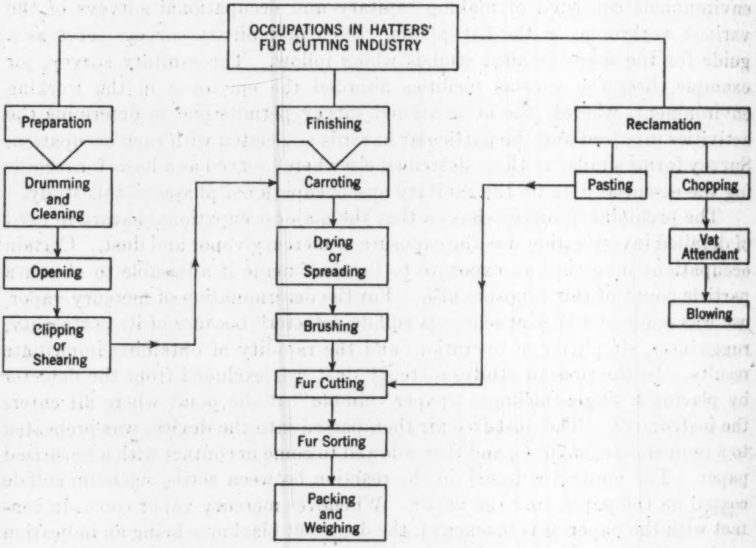
OCCUPATIONS IN HATTERS' FUR CUTTING INDUSTRY
Preparation
Finishing
Reclamation
Drumming and Cleaning
Carroting
Pasting
Chopping
Drying or Spreading
Vat Attendant
Opening
Brushing
Blowing
Clipping or Shearing
Fur Cutting
Fur Sorting
Packing and Weighing

FIG. 1

Fig. 1 is a flow sheet depicting the major operations involved. Briefly, the preparation of hatters' fur consists of softening and cleaning rabbit pelts by rotating them in a drum for about one hour with a mixture of wet sawdust and sand. This softens the brittle pelt and cleans the fur. The next step is to remove the sawdust and sand from the pelts by rotating them in a wire-mesh drum. The skins are then delivered to the openers, who mount the reversed skin on an upright wooden fork and cut off the head, tail, and legs, and slit the hide. Next the long hairs are removed by a revolving cutter, designated as clipping or shearing.

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[16] Final Report of the Miners' Phthisis Prevention Comm., Union of South Africa, March 10, 1919.

Up to this point the workers have been handling raw (untreated) skins. The processes which follow are associated with the use of mercury in one form or another.

In the next operation, known as carroting, a mixture of mercury nitrate and nitric acid is applied to the tips of the fur with either a hand or revolving brush. The skins are then dried either at room temperature or slightly below 140° F in mechanical driers (this is known as white carroting); or they are dried in ovens maintained at temperatures of approximately 240° F (yellow carroting). After drying, the skins may be stored for several months, or sent immediately to the brushing department. After treatment with the carroting solution, the fur is matted and irregular, and must be smoothed. This is done by brushers, who take each skin and subject it to a stiff revolving brush several times. From the brushing operation the treated skin goes to the cutting department where the fur is removed from the pelt by high-speed revolving blades which shear the fur and at the same time shred the pelt. The sheared fur then passes on to a conveying belt to the sorters, who remove rough scraps of skin cut by the machine from the fur. It is then packed in paper bags, each containing five pounds and is now ready for the hatting departments.

In some plants certain reclamation processes are conducted. Small scraps of fur recovered in the opening or sorting operations, or purchased from furriers, are chopped into small pieces and treated in one of two ways—the pieces may be glued to strips of manila paper and from then on handled as a pelt, or may receive vat treatment. After digestion in a vat, the scrap fur is freed of moisture in a centrifuge, then passed into a series of blowers which act as elutriators and free the fur of loose material. Other operations include storage and shipping of fur, and general maintenance and supervision.

RESULTS OF THE STUDY

Preliminary Surveys.—Most of the thirty-six plants covered in the preliminary survey were in old frame buildings, many of them being establishments of the so-called "back shop" variety. Only a few were in brick buildings, and of the entire number only two could be considered well planned from the standpoint of control of any existing occupational hazards.

Although insanitary conditions may not be associated with ill health, it has long been recognized that the elimination of sources of uncleanliness in factories is conducive to the well-being and efficiency of the workers. For this reason, the general sanitary conditions of the thirty-six plants were observed, and were found, on the whole, rather poor. The data obtained showed that 57% were in either a poor or bad state of sanitation, 33% fair, and the remaining 10% good or excellent.

The data in Table 1 indicate the number of workers in each plant, and compare such figures with other industries in the United States. They show that there was a greater percentage of larger plants in the hatters' fur cutting industry than is the case in other industries. For example, 55% of the 210,959 plants covered in the 1930 United States Census employed less than ten persons, as contrasted with only 8.4% in the hatters' fur cutting establishments. In fact, 20% of the thirty-six plants in the fur cutting industry employed 100 or more persons, as contrasted with only half that percentage for other plants in a typical industrial area. This may be due to the small number of plants in the fur cutting industry as compared with either the typical industrial area or the United States Census figures. However, it does show that most of these plants employed more than the average number of persons found in other trades.

TABLE 1.—COMPARISON OF PERCENTAGE DISTRIBUTION OF HATTERS' FUR CUTTING PLANTS WITH OTHER INDUSTRIAL ESTABLISHMENTS IN THE UNITED STATES ACCORDING TO NUMBER OF WORKERS

Type of establishment	Number of plants	PERCENTAGE OF PLANTS WITH LESS THAN:							More than 100 workers
		10 workers	20 workers	30 workers	40 workers	50 workers	75 workers	100 workers	
All industries in the U. S.*	210,959	55.0	72.0	77.0	81.2	85.2	88.8	91.9	8.1
Typical industrial area†	615	48.7	64.1	73.4	77.0	80.7	87.0	89.8	10.2
Hatters' fur cutting industry	36	8.4	27.8	38.9	52.8	63.9	69.5	80.6	19.4

* Personal communication from U. S. Census, 1930. † *Public Health Bulletin No. 216.*

A comparison of industrial welfare provision in the hatters' fur cutting industry and plants in a typical industrial area is shown in Table 2. The number of plants covered in the industrial area was 615, as compared to 36 in the hatters' fur cutting trades. It is apparent that such facilities as medical care, first aid, etc., are rather limited in the latter, as compared with the plants in the typical industrial area. Only 5.5% of the employees in the fur cutting trades had the services of either a part-time or full-time physician, as compared to 15.3% in the typical industrial area. When one considers that even in the typical industrial area the welfare provisions are rather limited as judged by present standards, it is evident that the hatters' fur cutting industry has much to be desired in providing these facilities.

TABLE 2.—COMPARISON OF INDUSTRIAL WELFARE PROVISIONS BETWEEN HATTERS' FUR CUTTING INDUSTRY AND INDUSTRIES IN A TYPICAL INDUSTRIAL AREA

Industry	Number of employees	PERCENTAGE OF PERSONS WITH LISTED FACILITY				
		Physician	Nurse	First-aid room	Sickness records	Sick Benefit Association
Hatters' fur cutting	1,970	5.5	10.1	21.8	10.1	10.1
Typical industrial area*	28,686	15.3	34.1	48.5	40.0	29.4

* "The Potential Problems of Industrial Hygiene in a Typical Industrial Area in the U. S.," *Public Health Bulletin No. 216.*

The preliminary study disclosed that most of the employees had been in the industry ten years or longer, and a number for more than thirty years, which is evidence that the industry is a stable one from the standpoint of labor turnover.

Sanitary and Occupational Surveys.—As a result of the preliminary study of the thirty-six plants, it was possible to select five factories for detailed investigation. The first step in an engineering study of this type is the making of a sanitary and occupational survey. Data pertaining to general sanitary conditions, ventilation, illumination, and environmental conditions, as well as air-borne toxic materials, were recorded; and information on the various occupations and the particular activities and raw materials associated with these occupations were noted.

In the hatters' fur cutting industry there were fourteen major occupations. Table 3 gives the number of workers by occupation in the thirty-six plants and contrasts these data with similar information on the five plants studied in detail. It may be seen that 57% of the workers were males, and that the occupation in which the greatest number of persons is employed is that of sorter—580 of the 2,004. A comparison of Columns (5) and (6), Table 3, gives an indication of the representative nature of the five plants selected for this study. In nearly every instance the percentage of workers by occupation in the five plants compared well with the figures for the industry as a whole.

TABLE 3.—OCCUPATIONAL ANALYSIS OF THE HATTERS' FUR CUTTING INDUSTRY CONTRASTED WITH DATA SECURED FROM FIVE PLANTS STUDIED

Occupation	Number of Employees in the Industry			Percentage of Total in:	
	Males	Females	Total	The industry	The five plants studied
(1)	(2)	(3)	(4)	(5)	(6)
Openers	24	133	157	7.9	5.5
Drummers and cleaners	57	0	57	2.8	2.0
Skin Sorters	13	11	24	1.2	0.9
Clippers	188	0	188	9.4	11.2
Carroters	180	2	182	9.1	7.3
Dryers and spreaders	52	45	97	4.8	7.0
Pilers	14	0	14	0.7	1.6
Brushers	74	26	100	5.0	4.4
Pasters and stickers	0	42	42	2.1	....
Cutters	145	2	147	7.3	8.6
Sorters	0	580	580	29.0	36.0
Packers	7	3	10	0.5	0.6
Shippers	22	0	22	1.1	2.0
General utility*	143	0	143	7.1	7.9
Blowers and choppers	135	0	135	6.7	5.0
Supervisors	60	8	68	3.4	....†
Clerical	23	15	38	1.9	....†
Grand Total	1,137	867	2,004	100.0	100.0

* Includes machinists, blade grinders, carpenters, elevator men, etc. † Included in general utility.

Occupational Exposure to Mercury Vapor and Dust.—A summary of the occupational exposure of hatters' fur workers to mercury vapor and dust is presented in Table 4. Samples of mercury dust and mercury vapor were obtained simultaneously for each occupation and the results of the two separate analyses were added in order to determine the total exposure to mercury for each occupation.

A total of 124 samples of mercury vapor and dust were obtained for the study. Additional samples, taken to evaluate certain other factors (such as efficiency of ventilation and for a special study on the control of the mercury hazard), are omitted from this discussion. The occupations have been arranged in the order of magnitude of exposure to mercury vapor and dust, and it is quite obvious that the highest average exposure is for shippers, who are exposed to 7.2 mg of mercury per 10 cu m, and this high exposure may be attributed to the fact that they are employed in storerooms housing thousands of pounds of treated fur, which is constantly giving off mercury vapor.

TABLE 4.—OCCUPATIONAL EXPOSURE OF HATTERS' FUR WORKERS TO MERCURY VAPOR AND DUST IN FIVE PLANTS

Occupation	Number of samples	Number of workers	Average mercury exposure, in milligrams per 10 cu m	Occupation	Number of samples	Number of workers	Average mercury exposure, in milligrams per 10 cu m
Shippers	6	11	7.2	Clippers	9	61	1.5
Pilers	4	9	5.4	Chopped fur blowers	2	2	1.3
Blowers	13	24	4.6	Skin sorters	3	5	1.2
Cutters	13	47	4.0	White carrot dryers or spreaders	11	32	1.2
Sorters	26	197	3.8	Carroters	13	40	0.8
Blown fur packer	1	3	3.8	Openers	2	30	0.7
Brushers	8	24	3.1	Miscellaneous (office, machinists, etc.)	5	43	0.6
Fur chopper	1	2	2.8				
Drummers	4	11	2.5				
Yellow carrot dryers	3	6	2.0				
				Total	124	547	....

The next highest exposure was found for pilers, who are exposed to large quantities of fur skins treated with mercuric nitrate. These skins, after treatment, are subjected to a curing process consisting merely in allowing them to age over a period of several months.

The occupation of blower entails an exposure of 4.6 mg per 10 cu m due to the fact that the workers are subjected to the breathing of dust from fur which has previously been carroted. Cutting and sorting, which "go hand-in-hand," involve an exposure of 4.0 mg of mercury vapor, and dust, whereas the lowest exposure is associated with miscellaneous occupations, involving personnel in utility activities, who are exposed to 0.6 mg in 10 cu m of air.

In connection with the exposure to mercury vapor and dust, two interesting findings should be mentioned. The literature on mercurialism among hatters' fur workers is replete with statements concerning the severity of the hazard among carroters. The present study indicates that carroters are exposed to relatively small quantities of mercury, the findings as shown in Table 4 averaging only 0.8 mg in 10 cu m of air. A close study of the operations involved in carroting shows that these workers are subjected to the possible inhalation of mercury nitrate and nitric acid mist, generated during the application of the carroting solution to the fur with a hand or mechanical brush; and, apparently, the quantity of mist is not very great. In fact, mercury in the form of vapor or dust does not begin to appear until later processes, and in those workrooms where treated furs have been stored for a considerable time. The chemical reaction which takes place subsequent to the application of mercury nitrate to raw fur, and which releases mercury vapor, is not definitely known. It is believed, however, that the reaction is somewhat as follows:

$$\text{Fur} + \text{Hg}(NO_3)_2 + HNO_3 \rightarrow \text{Keratin nitrate} + HgO \cdot N_2O_5$$
$$\text{Fur} + O \cdot Hg \cdot N_2O_5 + H_2O \rightarrow Hg + \text{Fur oxide} + 2\,HNO_3$$
$$\text{Fur} + HNO_3 \rightarrow \text{Keratin nitrate}$$

The reaction is slow, but it is well known that more mercury vapor is given off as the carroted fur becomes older. That there is some basis for these theoretical equations is indicated by the results for shippers, who are exposed to rather large quantities of mercury vapor emanating from the treated fur piled in the stockrooms.

It may be well to emphasize the second phenomenon observed. Following the preliminary survey made in the thirty-six plants, the first thought was that the small "back shop" type of plant with its extremely insanitary conditions would be the one in which considerable mercury exposure would be found; but actual quantitative studies indicated that these plants had relatively small quantities of mercury vapor and dust in the air, as compared with the larger plants. This may be explained by the fact that the small plants do not allow stock to accumulate, but work intermittently as orders are received. For this reason, little mercury is handled from day to day, and practically no treated furs are allowed to accumulate in the factories. On the other hand, in the larger plants, considerable quantities of mercury are being used, and hundreds of thousands of treated skins and vast quantities of treated fur are present continually. In addition, large quantities of treated and cut fur stock are allowed to stand in the various workrooms, often where there should be no mercury exposure, such as those in which preliminary processes are conducted, as in the room of openers and drummers. The remedy is apparent, since by segregation it would be possible to eliminate exposure to mercury for those workers whose tasks do not entail the handling of any mercury compounds or treated stock.

The exposure of workers to mercury compounds in the hatters' fur cutting industry was found to vary from a trace to a maximum of 10.4 mg per 10 cu m in the stockrooms. The exact effect of such exposures are discussed in another report dealing with the medical phases of this study.[17]

The present study afforded an opportunity to determine the effects of inhalation of organic dusts on workers, for which determinations of the dust concentration were made with the Konimeter device. This instrument was used because the nature of the dust involved precluded the use of the standard impinger method.

Table 5 presents the results of 112 determinations of the dust exposure. The highest was for drummers, namely, 16 million particles per cu ft. It is apparent that hatters' fur workers are exposed to relatively low dust concentrations, as judged by past studies of dusts associated with respiratory diseases.

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[17] "A Study of Chronic Mercurialism in the Hatters' Fur Cutting Industry," by R. R. Sayers, P. A. Neal, R. R. Jones, J. J. Bloomfield, J. M. DallaValle, and T. I. Edwards, *Public Health Bulletin No. 234*, May, 1937.

Control Studies.—Three methods are used in the hatters' fur industry for the control of mercury vapor and fur dust—segregation, local exhaust ventilation, and general natural ventilation.

TABLE 5.—OCCUPATIONAL EXPOSURE TO DUST AMONG HATTERS' FUR WORKERS

Occupation	Number of samples	Millions of particles per cubic foot	Occupation	Number of samples	Millions of particles per cubic foot
Drummers	4	15.9	Brushers	16	4.2
Blowers	8	9.4	Carroters	13	4.0
Sorters	31	7.1	Clippers	6	3.1
Cutters	18	7.0	Spreaders	4	2.3
Openers	6	6.3	Shippers	4	1.8
Packers	2	4.5			
Total				112	....

Segregation.—Segregation is one of the most effective methods of control available. It has a definite application to those operators who are concerned with the handling of raw, untreated pelts—namely, openers, drummers, skin sorters, and clippers. Complete segregation of those employed in these occupations would free 21.3% of the workers from mercury exposure. Table 6 illustrates how exposure to mercury may be decreased by segregating drummers and clippers.

Local Exhaust Ventilation.—No data are available relative to the air movements required for the control of mercury vapor and fur dust by local exhaust systems. The efficiency of the air volumes used for the removal of mercury where exhaust equipment has been installed had never been studied. In fact, brushers, cutters, and blowers have been exhausted chiefly to eliminate the nuisance caused by the dust generated, and even in these instances, efficiency has generally been based on the visual improvement secured. However, exhaust systems, as may be seen by reference to Table 6, decrease considerably the mercury exposure.

TABLE 6.—EXPOSURE OF HATTERS' FUR WORKERS TO MERCURY VAPOR AND TREATED FUR DUST UNDER CONTROLLED AND UNCONTROLLED CONDITIONS

Occupation	Total Mercury Exposure, in Milligrams per 10 Cu M		Average air flow, in cubic feet per meter	Method of control
	Uncontrolled	Controlled		
Drummers	2.5	0.6	....	Segregation
Clippers	1.5	0.7	....	Segregation
Brushers	3.1	1.2	300	Local exhaust ventilation
Cutters	4.0	1.8	383	Local exhaust ventilation
Sorters*	3.8	1.7	383	Local exhaust ventilation
Blowers	4.6	0.7	2,000	Local exhaust ventilation
Pilers	5.4	Trace	....	Good natural ventilation
Storage workers and shippers	7.2	Trace	....	Good natural ventilation

* Depend on exhausted cutters.

The method of control used on brushing machines is illustrated in Fig. 2. The exhaust opening is directly behind the revolving brush and the air is compelled to enter upward from the front of the machine which is open. An average of 300 cu ft of air per min per machine was found to produce the improvement indicated in Table 6. The reduction shown was obtained in a plant in which more brushes were operated than in the plants without control. This fact also implies the presence of more carroted skins in the brushing department which are continuously giving off mercury vapor.

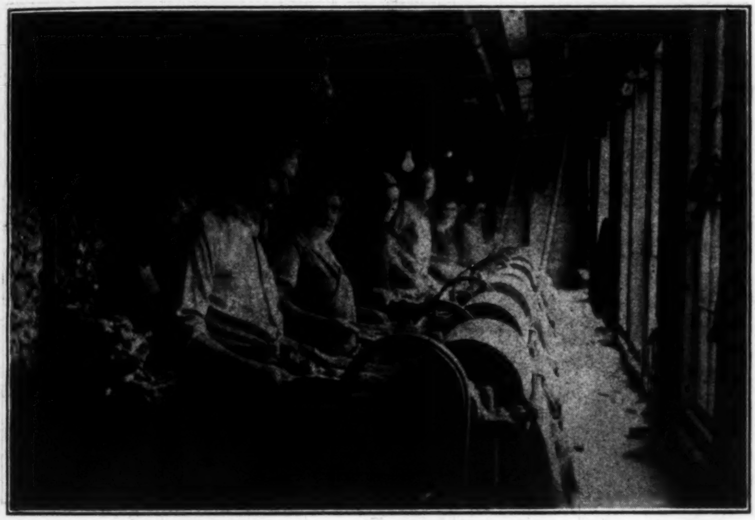

FIG. 2.—LOCAL EXHAUST VENTILATION FOR BRUSHING MACHINES

Cutting machines produce considerable dust due to vibration and to the fan action of the cutting blades, to which not only cutters, but also sorters, who are associated with them, are exposed. Thus (see Table 3), 36.3% of the workers in the industry are exposed to dust created by this operation alone. Moreover, these occupations are exposed to mercury vapor emitted by the large quantities of carroted skins and cut fur which are constantly present. The reduction through exhaust ventilation of mercury present in the air, both as vapor and as a constituent of the dust, is clearly shown in Table 6.

The manner of exhausting cutting machines is illustrated in Fig. 3. It will be noted that the hood is immediately behind the cutting blades, and draws air upward from the waste hopper below the machine. The sixteen cutting machines equipped with exhaust hoods each handled an average of 383 cu ft of air per min. The value was exceeded considerably on some machines close to the exhaust fans, but the increased air flows apparently did not interfere with the cutting operations.

Tables 5 and 6 indicate that blowers are associated with high mercury vapor and dust exposures, due to the presence of large quantities of treated fur and to the nature of the work done by blowing machines. To control the dust generated by one type of blower, an almost semicircular hood, concentric with the screened top of the blower, is used. The hood is constructed of varnished cloth in order to reduce the weight of the blower top, which must be removed periodically for cleaning. An adapter is fitted to the feeder end of the hood and connected to an exhaust leader. Air is thus compelled to move from the open end across the screened top of the blower and thence to the exhaust system. Ventilation studies made on this type of blower gave average air flows of approximately 2,000 cu ft per min.

Fig. 3.—Local Exhaust Ventilation for Cutting Machines

General Natural Ventilation.—This method of control was found in piling rooms and fur-storage basements. Because the storage of large amounts of treated fur is productive of considerable quantities of mercury vapor, it is essential that adequate air changes be provided. Good natural ventilation was found to be effective in one piling room and one storage room, in which outdoor air was forced to sweep through the rooms for twelve hours each day. Natural ventilation must not be considered as adequate unless there is a continuous positive sweep of fresh air.

Recommendations

The recommendations which follow are based entirely on what has been found to be good engineering practice in the industry itself. However, it so happens that the results of the medical studies suggest, for the present at least, a tentative limit of 2 mg of mercury in 10 cu m of air as a safe limit.[17] Reference to the results presented in Table 6 indicate that there are methods now in use in this industry which bring exposure below the threshold limit suggested by medical studies among the workers in this industry. Suggested remedies are as follows:

1. By segregating those operations handling untreated skins, approximately 23% of the workers may be removed entirely from the hazard of mercury exposure.

2. Local exhaust ventilation, properly designed and maintained for such operations as cutting, brushing, and blowing, will decrease the exposure for the workers in these occupations.

3. Good natural, or mechanical, ventilation decreased the quantity of mercury associated with the occupations of piling and shipping.

4. Since this study showed that in large plants considerable exposure may be attributed to the handling of mercury compounds in bulk, and to the storing of large quantities of treated fur, it is recommended that all treated material be removed from workrooms as quickly as possible and stored in well ventilated rooms.

5. Good "housekeeping" and general sanitation should serve to diminish the mercury concentrations in the workrooms. This study shows that mercury vapor is being generated constantly from treated fur. Hence, if treated fur skins and dust are allowed to accumulate, they will be a source of mercury vapor. It is recommended, therefore, that all floors, benches, and other objects on which dust may accumulate be swept and cleaned daily, either by wet methods or by vacuum. In addition, a complete general cleaning should be instituted once a week. In the larger plants it may be well to delegate the maintenance of protective equipment and the practice of good housekeeping to some responsible official who should make periodic inspections of all devices and methods used to minimize the mercury hazard.

6. As a result of this study it was shown that many of the plants have their processes very poorly arranged. Such practice is not only inefficient from the standpoint of production, but in this particular case serves to increase the exposure to mercury.

It is hoped that the methods outlined in the study of the mercury hazard in the hatters' fur cutting industry have served to indicate the rôle of the engineer in industrial sanitation studies. There are many other functions in studies of this nature, such as lighting, heating, safeguarding machinery, plant housekeeping, sewage disposal, water supply, cross connections, fire protection—in fact, those environmental conditions which form the basis of the work of the sanitary or public health engineer. It is not intended to minimize these latter activities because they have not been discussed in detail, but it is felt that such problems are fairly well understood.

Trends in Industrial Sanitation

The manifold functions of an engineer in the field of industrial sanitation, requiring as they do not only basic engineering training but also extensive preparation in public health and allied subjects, may raise the question in the minds of many as to whether there is a potential field to make it desirable from both a professional and a financial standpoint to undergo the necessary training for this phase of public health. Perhaps a few statements concerning this matter may serve to throw light on the subject.

It is known that there are approximately 50,000,000 persons in gainful pursuits in the United States, all of whom may be said to come within the scope of an industrial hygiene program. Of this number 15,000,000 workers are employed in industrial occupations, many of which are associated with hazards capable of being dealt with by an industrial sanitation program. Numerous specific occupational diseases are also associated with the industrial environment which may give rise to excessive morbidity and mortality rates in the industrial population. Of equal, or possibly of more, importance than specific occupational diseases is the fact that the incidence of other diseases such as tuberculosis, pneumonia, other respiratory diseases, and degenerative conditions is very high in the industrial population. Attention has also been directed from time to time to the fact that the life expectancy of the industrial worker is less than that of the nonindustrial worker. All of these facts indicate that industrial hygiene is indeed an important public health function—one which is receiving more and more attention, not only by public health workers, but also on the part of industry and labor.

Further proof of the realization on the part of the public that industrial hygiene is a vital concern of the state is attested by the rapid growth of industrial legislation taking place in the United States. Where several years ago only a few states recognized the need for compensating workers injured by occupational diseases, today (1939) there are twenty-one states which do so. As a result of this trend in industrial legislation and the realization on the part of those charged with the administration of such laws that it is necessary to have proper machinery for preventing industrial health hazards, the few years since 1936 have seen rapid strides made in the establishment of industrial hygiene services in many of the states and large cities. This growth has been made possible in part through certain provisions of the Social Security Act. Although large industrial establishments are beginning to provide their own industrial health facilities, one must not lose sight of the fact that most of the plants in the United States are small. In fact, according to the 1930 Census 97.5% of the plants employ less than 500 workers, and of the nearly 9,000,000 workers in manufacturing establishments 61% are employed in plants of this size. It is apparent, therefore, that if industrial hygiene is to be brought to the bulk of the industrial population it will have to be accomplished by means of a governmental agency. As the result of these various factors, the expansion of industrial hygiene work has been quite rapid, and where in 1936 industrial hygiene was confined to one or two agencies of the federal government and several state departments of health and labor, today (1939) one finds that there are nearly thirty industrial hygiene units in various states and cities.

A few years ago those faced with the administration of industrial hygiene would have been appalled by the paucity of data available on the control of health hazards in industry. Although considerable work remains to be done on this subject, sufficient data are now available concerning the effects of a large number of toxic materials and environmental conditions on the health of the worker, and what is more important, current knowledge of the methods for the control of many health hazards in industry has reached the stage where it may be applied successfully.

It should be apparent that all of these activities have resulted in a great demand for engineers capable of evaluating health hazards in industry and devising ways and means for their control. The burden of the problem resulting from the lack of trained personnel fell upon the U. S. Public Health Service for two reasons—its long experience in industrial hygiene work, and its administration of Social Security funds for this purpose. Realizing the urgency of this problem, the Division of Industrial Hygiene of the Health Service conducted two seminars of one month each for personnel selected to do industrial hygiene work in the state and city health departments. However, the demand for trained industrial hygiene workers still continues, and there is no reason to feel that this need will be satisfied for some time to come. Industrial establishments, insurance companies, universities, and state agencies are all calling for trained workers in this field of public health. It is apparent, therefore, that in view of the widespread interest in industrial hygiene, which in turn creates a demand for trained personnel, and because of the especial qualifications necessary to conduct industrial hygiene work successfully, adequate instruction should be available for those wishing to prepare themselves for a career in industrial hygiene. It is for these reasons that universities should give serious consideration to the institution of industrial hygiene courses as part of their regular or postgraduate curricula. Too much stress cannot be given to the necessity for such instruction in order that industrial hygiene workers may be prepared to deal with the problems concerning the health of workers in a manner that will produce substantial improvement and genuine progress in industrial life with the greatest efficiency and economy.

In order that an engineer may engage in industrial hygiene work he should be thoroughly trained in this field and should be well grounded in industrial processes. Such a person should have not only basic engineering training, but should be well versed, from a practical and theoretical standpoint, in the fields of microscopy, gas chemistry, physiology, and mechanics of ventilation, general sanitation, illumination, a knowledge of physical and chemical procedures, and industrial toxicology; and (most important of all), such an individual should have a broad public health background. It is apparent that a sanitary, mechanical, or chemical engineer, as such, does not fulfil these requirements exactly, but there is no reason why an individual with basic engineering training cannot, in time, prepare himself for work in industrial sanitation.

The duties and qualifications of an industrial hygiene engineer, which were adopted by the Committee on Industrial Hygiene of the State and Provincial Health Authorities, are as follows:

> "To determine under direction the necessity of making specific studies of particular industrial conditions; to conduct surveys and supervise studies

> of factory conditions predisposing to occupational diseases; to prepare comprehensive reports of findings with recommendations for control of occupational disease hazards; to supervise the work of field and laboratory workers; and to do related work as required.
>
> "The minimum qualifications call for graduation in chemical engineering, with 2 years' graduate work in industrial hygiene—to include ventilation, illumination, industrial toxicology, dust determinations; 3 years' experience in surveys and studies of industrial conditions for occupational disease control; or any equivalent combination of education and experience; familiarity with materials and processes used in industry; thorough knowledge of physical and chemical procedures for necessary determination of occupational disease hazards and of methods of control of these hazards; ability to recognize industrial processes and materials presenting potential occupational disease hazards; ability to enlist cooperation of plant executives, foremen, and laborers; initiative; tact; good judgment, and good address."

An attempt has been made to indicate the importance of the subject of industrial sanitation and the need today for trained workers in this field. The control of accidents and occupational diseases in industry has been shown to be largely an engineering problem, one which is varied and which has proved to be exceedingly interesting to those who have undertaken such work. It may occur to some that when many of the occupational diseases have been controlled the field for the engineer will become limited in scope and interest; but industrial sanitation, like many other endeavors, should continue to offer new opportunities and responsibilities. A field of industrial sanitation concerning which very little is known in the United States deals with the effects on health of recent trends in mass production. Little is known of the effect on the human system of work on the belt conveyor and the results of the "speed-up" in mass production industries. The subject of fatigue, which in all probability is associated with mass production, has been given little study. Still another subject in need of study is the effect of noise in industry on the health of workers. Another phase of industrial sanitation to which the engineer can make a definite contribution is that of housing, a subject that is receiving considerable attention. It is evident, therefore, that the field of industrial sanitation is not only virgin but will continue to be an interesting subject for many years to come—one which will test the ingenuity and ability of the engineering profession.

DISCUSSION

THEODORE HATCH,[18] ESQ. (by letter).—Mr. Bloomfield has indicated, in a broad way, some of the problems that confront the engineer in connection with the maintenance of health in industry and has illustrated these problems by a detailed discussion of the methods of analysis employed in the study of one particular health hazard—namely, mercury poisoning in connection with the preparation of hatter's fur. It should be pointed out that the procedures employed today in studies of this kind have been developed to a very large extent by Mr. Bloomfield and his associates in the U. S. Public Health Service.

To the sanitary engineer there is nothing basically new in the problems of industrial hygiene and sanitation since the objectives here, as in other phases of public health engineering, are to discover the nature of the relationship between health and environmental factors, to determine optimum values for health and safety with respect to hazardous conditions, and to develop control measures that will insure the maintenance of a healthful environment. The close cooperation between the physician and the engineer is essential in connection with this as well as other public health problems.

Sanitary engineers will also discover nothing basically new in the approach to the study of sanitation in industry as outlined by Mr. Bloomfield. The detailed techniques described by him, however, are probably not familiar to most sanitary engineers. He has limited his paper to one very small industry; but in doing so he emphasizes the great variety of the health problems that one encounters in industry and the corresponding variety of instruments and methods of analysis required for their study. He has emphasized further, however, that in all these problems the fundamental objectives in the investigation remain the same.

Work in industrial hygiene and sanitation in the United States extends over a period of more than 30 yr, but the most rapid expansion in this field has occurred in the last decade. In the beginning, the need for working tools tended to emphasize instruments, analytical methods, and laboratory experiments, but recently there has been a great increase in the number of governmental bureaus of industrial hygiene whose first interest must be to develop and maintain health in industry and not to develop instruments and methods of analysis. Thus, the practical exigencies of their duties may make it necessary for the workers in such bureaus to sacrifice elaborate scientific methods for procedures that will yield quick results. One of the best illustrations of this is seen in the problem of establishing standards of atmospheric cleanliness.

From the standpoint of the laboratory investigator, the best way to determine the toxic limit of an organic solvent or other industrial poison is by means of controlled animal and human experiments, so designed as to permit the complete exploration of the many different aspects of the problem. Considerable work has been done in this connection, but in practice one cannot depend upon costly and time-consuming experiments to supply the needed information with respect to the great variety of problems encountered in industry. Another approach to this problem is largely of an engineering nature and consists in determining the level of concentration of the toxic substance that can be secured by means of the best methods of control available to the industry and to fix upon this as the criterion of good operation. As control measures improve in efficiency, the permissible concentration would be lowered. There is precedent for this in all fields of public health engineering. The so-called Treasury Standard for public water supplies, for instance, establishes bacterial and chemical concentrations based almost entirely on the good practice in water purification rather than any direct epidemiological correlation between water pollution and disease. In the field of industrial sanitation this method has been employed with spectacular success in the gold mines in South Africa.

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With the development of the konimeter in 1916, dust surveys showed that the best practices in dust control reduced the mine dust concentration to 300 particles per cubic centimeter, and this value was adopted as a "figure of merit" or criterion of dust control for all mines. Since that time, the progressive decrease in dustiness has been followed by a corresponding decrease in the incidence of silicosis, and present experience indicates that the annual incidence of this disease has been reduced to a fraction of 1%. At the same time the average dust concentration has been reduced to below the standard established in 1916.

This engineering approach is probably the only one possible in certain industries. In foundries, for example, it is doubtful if one can show a direct relation between the degree of dust exposure, as measured by a dust survey, and the degree of lung damage, as measured by chest X-ray pictures and other examinations. This does not mean that a relationship does not exist but that it is overshadowed by other and more important factors, such as individual susceptibility. Moreover, the prevention of silicosis does not constitute the principal reason for dust control in the foundry industry. Foundry workers exhibit a higher mortality from other respiratory diseases, and dust control becomes a more general health measure. One may say, therefore, that the dust concentration in this industry should be maintained at the lowest possible level consistent with good engineering practice.

The conditions of sanitation and hygiene existing in industry have considerable influence upon the health record of the nation since nearly 10 million workers are exposed to the influence of their industrial environment for a third of each day. Industrial hygiene concerns itself, therefore (as Mr. Bloomfield states), with more than the control of specific poisons, which may be spectacular but not of great quantitative significance. Thus, silicosis and asbestosis are not widespread industrial diseases, but, on the other hand, the health of industrial workers is influenced to a very high degree by industrial dust in general. One of the important objectives of industrial sanitation is to introduce adequate measures of dust control into all dust producing establishments, regardless of the chemical composition of the dust. In the same way, problems of general ventilation, noise control, fatigue in relation to the environment, general plant sanitation, housing of the industrial worker, the effects of mass production, etc., all affect the health of the worker and should come within the purview of the engineer in this phase of public health work.

In conclusion, one may agree with Mr. Bloomfield "that the field of industrial sanitation is not only virgin but will continue to be an interesting subject for many years to come—one which will test the ingenuity and ability of the engineering profession."

Donald Francis Griffin,[19] **Jun. Am. Soc. C. E.** (by letter).—One source of a particularly obnoxious type of air pollution is the emission of hydrogen sulfide gas by decomposing sewage. The metropolitan area along the east San Francisco Bay in California is a notable example of a community whose air is being polluted from this source. In addition to the exposure to this type of pollution of a large residential area adjacent to a 20-mile waterfront, along which outfall sewers discharge, approximately 26,000,000 automobile passengers were exposed to it along the East Shore Highway during 1939.

There is the further possibility that the air, at least in the vicinity of the East Shore Highway, may be polluted with pathogenic organisms commonly found in raw or untreated sewage, through a combination of wind and wave action wafting droplets of infected water into the air.

The number of bacteria contained in such droplets "* * * depends upon the kind, the source, the conditions which lift them into the air, their survival rate, the air currents which transport, disperse and dilute them within the atmosphere, the rate of deposition and the physical and chemical conditions of their atmospheric environment."[20]

Very small droplets may evaporate before the force of gravity can precipitate them from the air; thus droplet or water-borne infection may change into air-borne infection. Under such circumstances short-lived pathogenic organisms may become air-borne.

In a test to determine the viability of droplet nuclei infection "* * * none of the four organisms typical of the intestinal tract, B. coli, B. dysenteriae Hiss Y, B. typhosus and B. paratyphosus A, were recovered at the end of the first day or at the end of eight hours when samples were taken after this time interval."[21] However, the interval between the time a droplet infected by one of the aforementioned organisms is wafted into the air and the time the infection becomes air-borne may be considerably less than eight hours. Perhaps additional tests using shorter time intervals are necessary to determine the viability of these organisms. This would appear especially desirable where large numbers of people are exposed near a source of droplet infection, as in the example of the East Shore Highway.

Assuming that infection may occur from such apparently short-lived pathogenic organisms as those mentioned, when air-borne, factors such as the number of people, and their proximity to sources of droplet infection, become important along with the viability of droplet nuclei infection. Present and future investigations may disclose that bacteria-infected air should be considered in relation to other factors as well as with contamination by organisms of the human respiratory tract.

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[19] Associate Planning Technician, National Resources Planning Board, Berkeley, Calif.

[20] "Preventive Medicine and Hygiene," by M. J. Rosenau, Ed. 6, 1935, p. 909.

[21] "Air-Borne Infection," by W. F. Wells and M. W. Wells, *Journal*, Am. Medical Assoc., 107, November 21, 1936, p. 1700.

Gordon M. Fair,[22] **M. Am. Soc. C. E.** (by letter).—Professor Phelps has given a stimulating and comprehensive survey of air sanitation, a field of engineering endeavor which, in its relation to the preservation and promotion of public health, has been gaining in recognized importance during the last few years. The future of air sanitation seems most promising. The sanitary control of the air, as outlined by Professor Phelps, offers broad and varied opportunities for engineering research and practice. In many respects it remains one of the untilled fields of public health. The engineer is certain to play an important rôle in the tillage of this field, provided he is willing to depart from classical engineering disciplines in a manner similar to the departure from routine civil engineering by the founders of water sanitation.

Three factors in air sanitation are of sufficiently broad significance to be added to those included in Professor Phelps' survey: (1) The control of odors; (2) the conditioning of air under abnormal pressures; and (3) the modification of the illumination of enclosed spaces.

Pollution of the outdoor atmosphere by odors has its source in industrial activities, including the treatment and disposal of sewage, industrial wastes, garbage, and refuse, as well as in other human activities such as the combustion of fuel for heating and transportation. The air of confined, occupied spaces may be rendered odoriferous by industrial and household processes and by the normal physiological processes of the occupants. Although odors are not known to be harmful, they do create discomfort and are important factors in air sanitation. The control of body odors may indeed be one of the limiting conditions in the ventilation requirements of closely occupied spaces such as school rooms, public vehicles, and assembly halls. Under the stimulus of odor control in water supplies, something has been learned about the measurement of odors, but much remains to be discovered about the differentiation of odors and their removal from the atmosphere by means other than their dilution to threshold values. The physiological and pathological response to odors also requires further investigation.

The return from high pressures to normal atmospheric conditions is frequently accompanied by disorders known as compressed-air illness, or caisson disease. These disorders are caused by the liberation of free nitrogen into the tissues and blood and the formation of bubbles of the gas which may block circulation. Similar phenomena may be associated with the entrance into low pressures from normal atmospheric conditions. High pressures are generally associated with industrial occupations such as diving, the sinking of caissons or shafts, and tunneling. Low pressures may be encountered in high-altitude flying and possibly in the treatment of certain pulmonary diseases. In each instance, the change that produces the disorder may be tempered by suitable engineering means. There is also a possibility of modifying the quality of the air itself within the abnormal atmosphere.

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[22] Prof. of San. Eng., Harvard Graduate School of Eng., Harvard Univ., Cambridge, Mass.

Although the control of the indoor air will probably remain very largely a matter of adjusting the temperature, humidity, and movement of the air and of cleansing the air supply, it is conceivable that sources of light, more closely akin in their radiations to sunlight and perhaps even more abundant in selected wave lengths, may play a part in the newer air sanitation. Such radiations are not generally emitted by present-day sources of illumination. Professor Phelps has mentioned the use of ultra-violet light for the disinfection of air but has not considered the wider application of beneficent radiations.

He ends his survey with the statement that "it is to the public health engineer that the future must look for guidance and responsible control" in air sanitation. This is a thought-provoking conclusion of importance to members of the Sanitary Engineering Division of the Society, as well as to sanitary engineers in general. It naturally raises the question as to whether it is practicable for sanitary engineers to encompass all of the branches of sanitation. Will they become "masters of none" if they make the attempt, and will they obstruct the wider usefulness of engineers in public health work? (The writer is unable to differentiate between "sanitary engineering" and "public health engineering." As Lemuel Shattuck's Sanitary Commission of Massachusetts stated in 1850: "The word 'sanitary' means 'relating to health.' " This sentence is amplified in a footnote of the Commission's Report from which the following may well be repeated. "We would divest our subject of all mystery and professional technicalities; and as it concerns everybody, we would adapt it to universal comprehension, and universal application." Has the word "sanitary" acquired a different connotation since Lemuel Shattuck's day?)

As previously stated, air sanitation as a field for study and professional activity has come to the front within the last few years. The training of many state and municipal sanitary engineers in this branch of sanitation appears to have been inadequate, as evidenced by the fact that the U. S. Public Health Service has found it expedient to conduct short courses in industrial hygiene for such engineers. The writer knows of only a few universities in which rigorous training in air sanitation is available to engineering students. As stated by Professor Phelps, engineering design connected with air sanitation has been done in the past by groups of engineers who, however willing to meet requirements that health authorities may have prescribed, have lacked the background of "knowledge of the physical, chemical, bacteriologic, physiologic and toxicologic properties of the special types of impurities" on which the sanitary control of the air must be founded. Basic research by engineers appears to have been limited or spasmodic.

The group of engineers that has been widely interested in emphasizing the public health aspects of certain of its engineering activities has been the civil engineering group. This is attested by the existence of the Sanitary Engineering Division in the Society. It follows that, in the past, most sanitary engineers have dealt more particularly with water sanitation. This situation is natural, since opportunities for the application of specialized public health knowledge have been most favorable in engineering endeavor connected with water supply and sewerage. The incentive for study, in a practical world, must be provided of necessity by the opportunity for gainful occupation.

It has been the writer's experience that, in the normal period of study commonly available to the training of sanitary engineers, it is not possible to include too large a group of disciplines without sacrificing thoroughness. There are certain fundamental disciplines, lying outside the realm of engineering proper, that are not shared alike by the engineer who is to fit himself for the sanitation of water and the engineer who is to become competent in the sanitation of air. Notable among these disciplines is human physiology, or ecology, coupled with toxicology; these are subjects essential to the intelligent and intellectual appreciation of air sanitation, but not so important in water sanitation. Furthermore, there must be a difference in emphasis upon certain basic engineering sciences, as well as biological and chemical sciences, in their relation to water on the one hand and air on the other.

Given sufficient time for study, it is possible to train men to become qualified in both water sanitation and air sanitation. The writer does not believe, however, that in the ordinary management of human affairs an adequate number of men can hope to maintain enduring intellectual and practical contacts with both these branches of sanitary engineering. In the long run, expertness in one or the other branch must be sacrificed if degeneration in both is to be avoided. It would seem wise, therefore, to recognize that a diversity of interest and practice in sanitary engineering is a norm of the profession.

Only two branches of sanitation have been contrasted—namely, water sanitation and air sanitation. There are others: Milk, shellfish, and food sanitation; insect and rodent control; housing; illumination; noise suppression; and the disposal of solid municipal refuse. It stands to reason that engineers whose main interest lies in water supply and sewerage will continue to be recruited chiefly from the ranks of civil engineers and chemical engineers, whereas engineers who deal more particularly with the sanitation of the air may more probably possess a background of mechanical or chemical engineering. Civil, mechanical, electrical, architectural, and agricultural engineers may share in other branches of sanitation. Sanitation, indeed, must enter in some measure into all engineering practice because engineering is essentially the control of the human environment for the purpose of creating human comfort and well-being. A natural cleavage will not be prevented by a "dog in the manger" attitude based on historical priority or superficial knowledge.

It is to be hoped that American engineers will defend and preserve their heritage of participation in public health activities by continuing to acquire the fundamental, non-engineering knowledge necessary for the comprehensive pursuit of their sanitary work; furthermore, that by participation in research they will continue to play a significant part in the advancement of this fundamental knowledge in its relations to engineering practice. It is to be hoped, finally, that the diverse groups of engineers engaged in the various branches of sanitation will be bound together as sanitary engineers by a unifying spirit of public service and a common interest and expertness in the protection and promotion of the public health.

H. G. Dyktor,[23] **Assoc. M. Am. Soc. C. E.** (by letter).—Mr. Bloomfield has ably covered the subject of the problem of industrial sanitation. There is one point, however, that should be stressed particularly, since the purpose of the Symposium seems to be to arouse the interest of all engineers in the necessity for industrial sanitation—namely, the extent to which engineers in industry can cooperate with the industrial hygienist without requiring the specialized knowledge and experience of the latter. This cooperative effort should be motivated by the desire to remove hazards to the health of the plant worker. Industry realizes that it is good business to have healthy workers, and engineers should realize it also. Moreover, only common sense and its intelligent application are necessary to effectuate this cooperation.

The industrial hygienist requires cooperation from plant officials on the following three items which are basic to all plants. This does not mean that they are the only ones but it does mean that they are the simplest, least expensive, and of general and immediate benefit both to the plant and its workers: (*a*) Housekeeping, (*b*) general sanitation, and (*c*) maintenance of protective equipment.

(*a*) *Housekeeping.*—By "housekeeping" is meant the systematic arrangement of the equipment, materials, and products in a plant. Under this heading the following items are to be found: Cleanliness, orderliness, and such regular routines as wet or vacuum sweeping of floors and joists, cleaning windows and electric light bulbs, etc. It is logical to insist on a clean and orderly plant because such a condition will be conducive both to safety and to a sanitary environment for the workers. It is also the base line from which further and more involved improvements may originate.

(*b*) *General Sanitation.*—The provision of a safe water supply, proper sewage disposal, and the absence of any cross-connection between these two are included under "General sanitation." It includes also the provision of sanitary and approved drinking water dispensers (no common cups or glasses), individual towels (either paper or cloth, but not common towels), satisfactory and adequate toilets and urinals (properly separated for sex) etc. Welfare rooms, such as locker rooms, wash rooms, etc., should be light and clean. Illumination and ventilation of the plant should also be satisfactory and at least should meet minimum requirements.

(*c*) *Maintenance of Protective Equipment.*—Mechanical and protective equipment is expensive and should be protected by proper maintenance. Moreover, its efficiency depends on such maintenance. Sporadic and intermittent efforts in that direction are not as desirable as a carefully planned and regularly executed procedure. For instance, battered hoods should be reshaped to their original form, ducts should be examined for clogging, broken seams and other openings, all of which interfere with, and reduce the efficiency of exhaust equipment. Likewise collectors must be emptied and cleaned at regular intervals. Personal equipment such as respirators should have their filters, etc., replaced when filled, so as to reduce unnecessary resistance to breathing, and holders should be cleaned and kept in dust-free places. Positive pressure helmets should have their reducing valves checked; compressed air should be filtered to remove oil and should be analyzed periodically for carbon monoxide. Many other precautionary measures could be listed.

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[23] Chf. Engr., Bureau of Industrial Hygiene, State Dept. of Health, Lansing, Mich.

The foregoing brief discussion contains no sanitary measures that could not be accomplished by the average engineer and yet, if these items were attended to a great load would be removed from the shoulders of the industrial hygienist who could then spend the time thus saved on other matters requiring his specialized knowledge and experience.

Gilbert H. Dunstan,[24] Assoc. M. Am. Soc. C. E. (by letter).—So much development in the field of sanitation and public health has occurred during the past decade that this Symposium is of great value in focusing attention upon one important phase. Professor Phelps has mentioned the pollution of the outdoors atmosphere, and its effect upon health. From the economic standpoint, such pollution may have other implications, one of which is flood control. Denuding hillsides of all vegetation by the fumes from smelters may require costly flood-control structures to prevent damage, in some cases even to the smelter itself. This is the case in Kessler Canyon, Utah.[25]

Mr. Bloomfield has made a valuable contribution by indicating the method of attack in an important problem of industrial hygiene. Engineers must take an increasing interest in this type of work. Mr. Bloomfield quotes the duties and qualifications of an industrial hygiene engineer, which were adopted by the Committee on Industrial Hygiene of the State and Provincial Health Authorities. These indicate graduation in chemical engineering as a qualification, followed by at least two years of graduate work in industrial hygiene, plus three years of experience. The writer would raise the question of whether or not a somewhat better sequence of studies might not be arranged if the man graduated in sanitary engineering and followed this by two years of postgraduate study. In any case, the breadth of training required seems to indicate at least six years of formal training.

Certainly the two terms "sanitary engineering" and "public health engineering" may be considered to be more or less synonymous; yet there seems to be a tendency to give a slightly different meaning to each of them. Recent studies of literature relating to training in these fields seem to show a tendency to consider the sanitary engineer as the one who is concerned chiefly with water supply and treatment, sewerage and sewage treatment and disposal systems, and stream pollution. The public health engineer is concerned with these factors largely in a supervisory way; but in addition he is interested in rural sanitation, supervision of food supplies, and many other problems of a somewhat minor nature, involving engineering to some extent. He must deal with the public to a great extent, and his work is more closely related to that of the medical profession. The industrial hygienist is closer to the public health engineer than to the sanitary engineer under this distinction. Only one institution grants degrees designated in these two fields, and it is the writer's belief that the foregoing distinction is indicated by the two curricula as offered.

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[24] Asst. Prof. of San. Eng., Univ. of Alabama, University, Ala.

[25] "The Barrier System of Flood Control," by L. M. Winsor, M. Am. Soc. C. E., *Civil Engineering*, October, 1938, p. 675.

However, at the present time the two terms are used loosely, and only time will tell whether any real distinction between the two fields will develop.

Charles Lundy Pool,[26] M. Am. Soc. C. E. (by letter).—In common with others competent to judge, the authors of this Symposium agree on certain fundamentals. A vast amount of new, useful knowledge is available to the sanitary engineering profession, the application of which promises both economic and public health benefits.

Some engineers sound a warning that the individual must beware lest attempted expertness in the fields of sanitation of waters and hygiene of air lead to accomplishment in neither. They recognize, however, that the profession is not limited by the limitations of its individual members and can effectively embrace diversified fields of endeavor without this danger, provided the fields are related. Others express the principle that in many cases the engineering approach is the only one to set a standard for improved conditions. Advances which new techniques make possible are often put into effect to better man's environment, although their relationship to his well-being cannot be measured in the maze of variables present, not the least of which is variable susceptibility of the individual.

Mr. Bloomfield testifies to the fact (see heading "Trends in Industrial Sanitation") that within the past few years "current knowledge of the methods for the control of many health hazards in industry has reached the stage where it may be applied successfully." He also presents conclusive data to impress the need of application of the new knowledge. Professor Phelps' review of the branches of science which now have much to contribute includes air bacteriology which has enjoyed a renaissance under the "midwifery" of W. F. Wells. Sanitarians may now compare airs as to their relative contamination so as to supplement the present physiologico-thermal guides by bacteriological controls. Professor Phelps also states (heading, "Industrial Conditions"), "It is believed that the public health engineer must be prepared, with the necessary collaboration from the associated sciences, to assume ultimate responsibility and control in all such cases."

Now, what can be done to harness all this benevolent sporadicity of which the foregoing examples are but indicators of a vast ground swell? Industrial air sanitation has made rapid strides the past several years, and has been integrated into the work of health departments. The latter is not true of air sanitation of communities or public buildings. The majority of state health departments now have industrial hygiene engineering services.

If stream-pollution abatement programs had been left to progress only by the "fits and starts" which common law or damage suits might have stimulated, without the coordination of programs by state sanitary engineers, the art of wastes disposal would indeed be in a sorry state today, and its practice worse. Air sanitation of public places likewise needs to be ordered into the program of all state departments of health. This is particularly true today in view of the evolution of these departments to encompass more and more all applications of sanitation whether subject to immediately demonstrable health significance or not.

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[26] Chf. Engr., Div. of San. Eng., State Dept. of Public Health, Providence, R. I.

The work of the sanitary engineering, and of the industrial hygiene engineering, services should be closely coordinated to effect this program. The limitations of the individual must not be allowed to impede the progress of the profession. There is no reason why these two synonymous groups of sanitary or hygiene engineering cannot learn to work together in the same manner as water works engineers and sewerage works engineers, or bridge engineers and highway engineers.

In each state department the engineering service should prepare itself to evaluate all the programs of all branches of environmental hygiene and sanitation in the state, no matter by what state or local agency these are conducted. In this manner annual engineering reports to Commissioners of Health could be made comprehensive for the place which sanitation plays and should be made to play in the state economy. The administrative review of the status and adequacy of current endeavors need not require an expert qualified to apply the details of each branch of sanitation but, of course, must be conducted by one broadly grounded in the science of public health. Expert assistance in branch specialties may be had from the U. S. Public Health Service in the conduct of such evaluational surveys.

This general program is broached in this discussion on air sanitation because in many instances the topic could well be made the point of departure from an old lethargy. Of timely interest in the program into which air sanitation can fit is the following communication, addressed to all State Health Officers by the Surgeon General of the U. S. Public Health Service:

> "On February 1, 1940, a reorganization of the work of the Sanitation Section was effected with the object of transferring to that Section and its district engineer offices all environmental sanitation functions not already assigned to it.
>
> "The following units have been established, or will shortly be established, in the Sanitation Section: (*a*) Water Pollution Control (treatment of sewage and industrial wastes); (*b*) Rural and Suburban Excreta Sanitation (structures not connected to sewer systems); (*c*) Water Supply Sanitation; (*d*) Milk and Milk Products Sanitation; (*e*) Food Sanitation; (*f*) Garbage Disposal and Rat Control; (*g*) Recreational Sanitation; and (*h*) General Sanitation.
>
> "In accordance with the above reorganization the Milk Specialists and the Community Sanitation Consultants have recently been made responsible to the District Engineers, who are in turn responsible to the Chief of the Sanitation Section."

Note that the pattern is being set by the district engineers' offices of the federal service into the program of which state engineering services will need to be integrated. Note that "all environmental sanitation functions" are assigned to the Sanitation Section. The next unit that should be established beyond those listed, at first in industrial districts, should be one specifically for air hygiene. When the demand is felt there is no doubt but what this may be had.

Since state sanitary engineering functions are becoming so diversified it is of additional importance that federal district experts be stationed where they may be available on call to assist the state engineers in their salient duty of providing a clearing house for the advanced knowledge of all branches of sanitation. Especially is this true of expert advice on air sanitation in view of its newness and consequent imperfect dissemination.

It is most important that the entire sanitary engineering profession digest the significance of the trends and act as a body to influence and promote an ordered and purposeful evolution.

J. J. BLOOMFIELD,[27] ESQ. (by letter).—Messrs. Hatch, Griffin, Fair, Dyktor, Dunstan, and Pool have enhanced the original material in the papers presented by Professor Phelps and the writer by their excellent discussions and have brought out many important points bearing on the subject of the rôle of the engineer in air sanitation. The writer will not attempt to comment on their discussions but will confine his closing remarks to two points which, perhaps, need further emphasis. These points concern themselves with the preparation of an engineer for industrial hygiene work and the rôle of health department engineers in this field.

In the paper the writer discussed the type of training that an engineer should undergo in order to qualify himself for work in industrial hygiene. He also outlined those duties and qualifications of an industrial hygiene engineer which were developed by the Committee on Industrial Hygiene of the State and Provincial Health Authorities. This latter committee specified that the individual be a graduate in chemical engineering. However, in his version of the preparation of an engineer for industrial hygiene work, the writer stated that an individual with basic engineering training—in sanitation, mechanics, or chemistry—could become qualified (with special preparation) to conduct work in industrial sanitation. In view of the fact that industrial hygiene as practiced today includes all phases dealing with man's working and living environment, it would seem that an engineer with basic training in public health procedures would be best qualified to undertake this type of work, with additional training in those subjects outlined in the paper.

The other point which the writer desires to emphasize concerns itself with the necessity for engineers in public health work to recognize that industrial hygiene is as much a function of their activities as of the physician, nurse, or other public health worker. Industrial hygiene attempts to improve the hygiene of the individual and the environment in which he works and lives. The latter function belongs within the sphere of the engineer. Unfortunately, in the past, and to some extent even today, many engineers in public health work have confined their efforts to the control of the water supply, sewage disposal, foods, and milk, but have overlooked other problems becoming significantly important of late, such as air pollution and housing. Professor Phelps has defined public health engineering as the art of directing the forces and activities of Nature to the protection and improvement of the public health. This is a

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[27] Sanitary Engr., U. S. Public Health Service, Washington, D. C.

sound definition and engineers in public health work should seize the opportunities now before them and expand their activities to embrace all the problems in environmental hygiene, so as to include not only man's environment at home but also at his work place. Every engineering division in a health department—in a state, city, or county—should have personnel on its staff prepared to deal with the manifold problems of sanitation in industry. It is only by such an approach that engineers in public health may rightfully consider themselves as fulfilling their functions.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 2101

EFFECTS OF RIFLING ON FOUR-INCH PIPE TRANSPORTING SOLIDS

By G. W. Howard,[1] Jun. Am. Soc. C. E.

With Discussion by Messrs. Fred R. Brown, David L. Neuman, R. Y. Newell, Jr., and G. W. Howard.

Synopsis

Tests to determine the effects of rifling installed in 4-in. pipe upon the transportation of mixtures of water and material are described in this paper. Results were compared with those from 2-in. pipe in an effort to discover any similarity between the transportation characteristics of each pipe such that the principle of these smaller pipes could be applied to larger pipe. Tests for the development of the optimum design of rifling for pipes in which sand was the transported material constituted the major part of the testing program. The rifling was tested later with silt, clay, and pea gravel used as the transported materials.

An attempt is made to show that the results obtained through the study of 2-in. and 4-in. pipe can be extended to include pipes having a larger diameter; and further, that, under certain conditions, dredging technique of the present day can be improved considerably by the proper use of rifling in the discharge line.

Introduction

Material transported in pipe lines has been found to concentrate in the lower part of the pipe.[2] Knowledge of this phenomenon caused the Memphis (Tenn.) Engineer District, in 1913, to experiment with a mixer to provide uniform distribution of the material over the cross section of the pipe (see Fig. 1). The mixer that was tested consisted of a diametrically warped plate, with its upstream edge horizontal, fastened in the pipe. The pitch of this mixer was 10 pipe diameters and its length was 5 pipe diameters. Fastening this plate in the pipe effected a division of the contents into two parts—an

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Note.—Published in November, 1939, *Proceedings*.

[1] Capt., Corps of Engrs., U. S. Army, Fort Belvoir, Va.

[2] *Transactions*, Am. Soc. C. E., Vols. LVII (1906) and 104 (1939), pp. 403 and 1334, respectively. See also *Professional Memoirs*, Corps of Engineers, U. S. Army, March–April, 1915.

upper and a lower half. In passing through the mixer, the two halves of the contents reversed their positions: The upper part became the lower part, and the lower part became the upper part. A horizontal plate, having a length of 3.8 diameters, was added to prevent the material from being carried down to the bottom of the pipe as it emerged with a spiral motion; the purpose of this plate, then, was to deliver the material with its motion parallel to the axis of the pipe, but with its position changed from the lower half to the upper half.

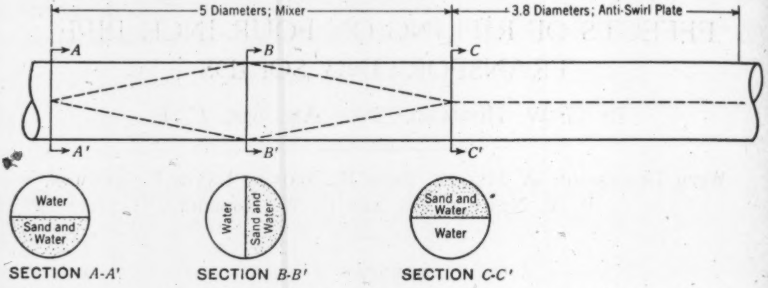

Fig. 1

Tests on this mixer showed that it was successful in locations where pure sand was encountered—that is, in material where no tree stumps were embedded. The reduction of half the effective area of the pipe for passing stumps caused considerable difficulty in blocking the line. As a result of this limitation, tests on the mixer were abandoned. However, investigation on the warped plate mixer led to a comprehensive series of experiments at the U. S. Waterways Experiment Station, during the period 1935–1937, to develop a mixer for installation in pipe lines.

Program of Experiments

Two separate series of tests were performed. The first series included tests on 4-in. pipe to determine the optimum design of rifling for use in a pipe line transporting sand. Results obtained from this design were checked with those from tests on 2-in. pipe. The second series included tests to determine the characteristics of the optimum rifling when materials other than sand were transported. For these tests the materials used were (*a*) silt, (*b*) clay balls, and (*c*) pea gravel.

Set-Up for Experiments

The apparatus[3] used for these investigations is shown diagrammatically in Fig. 2. Briefly, the apparatus consisted of a circulating system wherein water, or water mixed with material, was pumped from a sump to a variable head tank, from which it was discharged through the test pipe line back into the sump. Manometers were placed above and below the section of pipe being

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[3] For a complete description, see *Transactions*, Vol. 104 (1939), p. 1334.

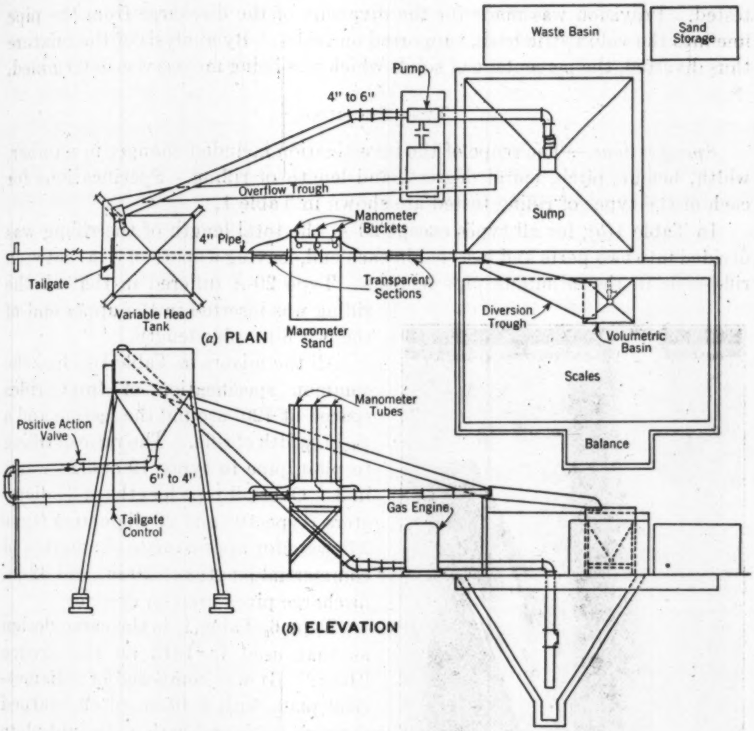

Waste Basin
Sand Storage
Pump
4" to 6"
Overflow Trough
Sump
Manometer Buckets
4" Pipe
Tailgate
Transparent Sections
Variable Head Tank
Manometer Stand
Diversion Trough
Volumetric Basin
(a) PLAN
Scales
Manometer Tubes
Positive Action Valve
6" to 4"
Balance
Gas Engine
Tailgate Control
(b) ELEVATION

FIG. 2.—APPARATUS USED ON TESTS FOR SAND AND GRAVEL

TABLE 1.—SPECIFICATIONS FOR RIFLING

(a) FIRST SERIES

Type	Number of rifles	Dimensions of Rifles, in Inches			Spiral, in degrees	Total length, in pipe diameters
		Width	Height	Pitch		
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	1	1/8	1/2	40	180	5
2	1	1/16	1/2	40	180	5
3	1	3/16	1/2	40	180	5
4	1	1/8	1/4	40	180	5
5	1	1/8	3/4	40	180	5
6*	1	...†	...†	40	180	...†
7	1	1/8	1 1/2	20	54	1 1/2
8	3	1/16	3/4	40	360‡	10
9	3	1/16	3/4	20	360‡	5
10	3	1/16	3/4	40	180	5
11	3	1/16	3/4	20	180	2 1/2
12	3	1/16	1/2	40	360‡	10
13	3	1/16	3/4	40	54	1 1/2
14	3	1/16	1/4	120	....§	43 1/3
15	3	1/16	3/8	80	....§	43 1/3 86 1/4

(b) SECOND SERIES

Type	Dimensions of Rifles, in Inches		Total length, in pipe diameters
	Height	Pitch	
(1)	(4)	(5)	(7)
16	1/2	40	2 1/4
17	3/4	40	2 1/4
18	3/4	30	2 1/4
19	1/2	40	6 1/2
20	1/2	40	4 1/2
20-A	1/2	40	5 1/2
21	1/2	60	6 1/2
23	1/2	60	4 1/2
24	1/2	40	8

* Design in original field tests, in 1913.

† Diametrical plate; 15 in. of straight plate at end of warped section.

‡ Rifles spaced at 120° around the pipe.

§ Continuous.

tested. Provision was made for the diversion of the discharge from the pipe line into the volumetric basin supported on scales. By analysis of the mixture thus diverted, the percentage of solids which was being moved was determined.

TEST SECTIONS

Specifications.—The scope of the investigation included changes in number, width, height, pitch, spiral degrees, and length of rifling. Specifications for each of the types of rifling tested are shown in Table 1.

In Table 1(*b*), for all types except 20-A, the total length of the rifling was divided into two parts and inserted in each end, leaving a space of 3 in. between rifle ends in the adjoining pipe lengths. Type 20-A differed in that all the rifling was inserted in the upper end of the 6-ft 8-in. pipe length.

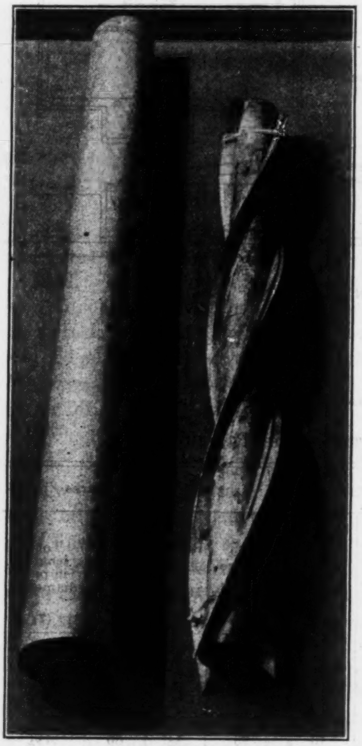


FIG. 3

All the mixers in Table 1(*b*) had the common specification of three rifles spaced at 120° around the section and a $\frac{1}{16}$-in. width of rifle. The ratio of rifling to plain pipe in types 19 and 24 was 1 in 3. Over-all pipe lengths of 24 diameters (type 19) and 20 diameters (type 24) were for approximate simulation of commercial lengths of 20-in. and 32-in. discharge pipes used on dredges.

Type 6, Table 1, is the same design as that used in 1913 on the dredge "Iota." It was composed of a diametrical plate with a 40-in. pitch, warped through 180°, and with a straight plate on the end for the next 15 in. In types 8, 9, 12, and 13, the rifles were spaced at 120° around the pipe. The single rifle in type 7, Table 1, was made like a piston so that it could be removed from the line or set at any height. In types 10 and 11, the rifles were spaced at 60° around the pipe, the center rifle starting on the bottom of the pipe.

The rifles were spaced at 120° around the pipe in the case of types 12 and 13, Table 1; and in types 14 and 15, the height of rifles was $\frac{1}{2}$ in. for the first 10 diameters and $\frac{1}{4}$ and $\frac{3}{8}$ in., respectively, for the remaining length. Type 15 was tested twice, first at $43\frac{1}{2}$ diameters and next at $86\frac{1}{4}$ diameters.

Design and Construction.—In order to obtain a smooth finish inside the pipe after the installation of rifling, it was necessary that each rifle fit flush along the pipe, prior to soldering. The pipe itself was made from 20-gage sheet metal and the rifles were of 16-gage metal. Although it was possible to bend these strips of metal and obtain an approximate fit, the absence of an exact fit would permit a small amount of solder to enter beneath the strip and cause enough bulge to change the height. Therefore, each strip was designed as a segment of a cone with the distances along the upper and lower edges of the rifle as diameters. The slant height of this segment was the height of the rifle, and the total slant height of the cone was the radius used to describe the arc along which to cut the metal.

A metal core was used to hold the rifles in place during the process of soldering inside the pipe. Grooves on the outside of this core maintained the correct pitch and, after fastening one end of each rifle, the core guiding the strips into place was twisted through the section. Fig. 3 shows the arrangement used for installing the rifles immediately before placing the core inside the pipe.

PROCEDURE OF EXPERIMENTATION

The attempt to discover the optimum rifling led to the conclusion that tests should be conducted which involved the determination of the following: (*a*) The characteristics (head loss versus velocity) of a pipe, without rifling, carrying a sand and water mixture; (*b*) the characteristics of different designs of rifling when identical sand and water mixtures were transported; (*c*) the effect of changes in the size of the pipe upon its transportation characteristics; and (*d*) the characteristics of the optimum rifling when transporting mixtures of water and silt, water and clay balls, and water and pea gravel.

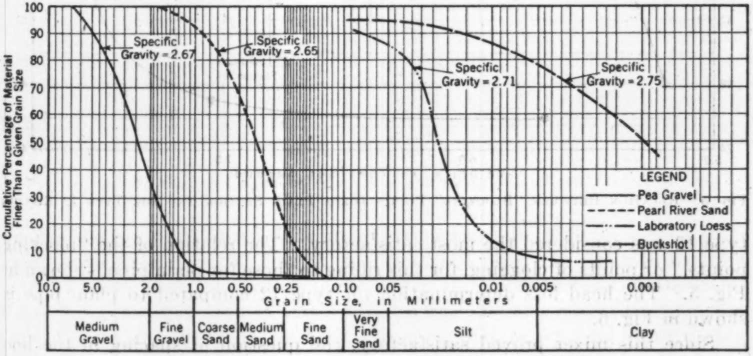


FIG. 4.—MECHANICAL ANALYSES OF MATERIALS TESTED

Determination of concentration, velocity, and pressure were the primary features in test operation. Solid concentration was determined from the weight of a known volume of the mixture, and pressures were measured with manometers connected by tubes to piezometers (the manometers were installed in the same location for all test sections, providing direct comparisons for the different types of rifling).

TEST RESULTS

Sand Tests, 4-In. Pipe.—The material used in the investigation was known locally as Pearl River sand. A mechanical analysis of the material is shown in Fig. 4. Investigations in 4-in. plain pipe showed that a velocity range of from 6.0 to 8.5 ft per sec covered all types of sand transportation in pipe lines.[3] Accordingly, this range was used for tests on mixers.

The initial investigations included tests from type 1 through type 12 (see Table 1(*a*)). In these tests the design of rifles was of primary importance—that is, the width, height, pitch, and spiral. Of the designs that were tested, type 12 was considered the most satisfactory. The relation of the "blocking points," or points of clogging, for this rifling to those for plain pipe is shown in Fig. 5. The head loss determination for type 12 compared to plain pipe is shown in Fig. 6.

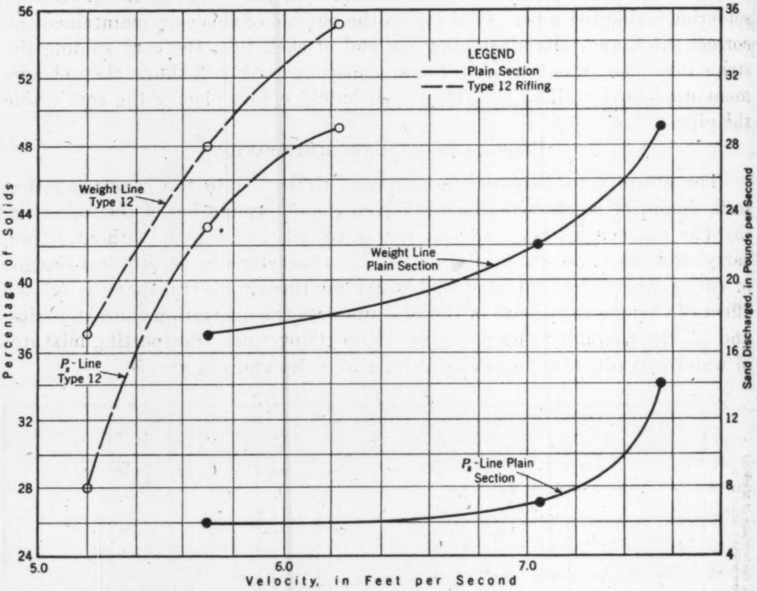


FIG. 5.—RELATION BETWEEN "BLOCKING" POINTS FOR A PLAIN SECTION AND FOR TYPE 12 RIFLING

Since this mixer proved satisfactory, the question of spacing in the line was the next to be considered. Numerous spacings were tested in an effort to determine which location would be satisfactory. The results of this investigation, for the optimum ratio of rifling to over-all pipe length, are presented graphically in Fig. 7. From these results it was determined that the length of rifling should be one-third the length of the over-all pipe section. Note that all velocity curves shown on this sheet are for center-line velocities. The dimensions of type 12 rifling are given in Table 1(*a*).

Investigation was next made of pipe lengths of 19½ and 24 diameters. (The length of commercial pipe has diameters of 32 in. and 20 in., respectively.) From this investigation a rifling was designed which was known as type 24 and had specifications as follows: 3 rifles spaced at 120° around the section; rifling ⅛ diameter in height, and 10 diameters in pitch; and length of pipe having rifling one third the length of the over-all pipe section (pipe sections tested were 19½ and 24 diameters long); this length was divided in two and inserted in each end of the pipe. The curves for head loss for this rifling and plain pipe are shown in Fig. 8(*a*).

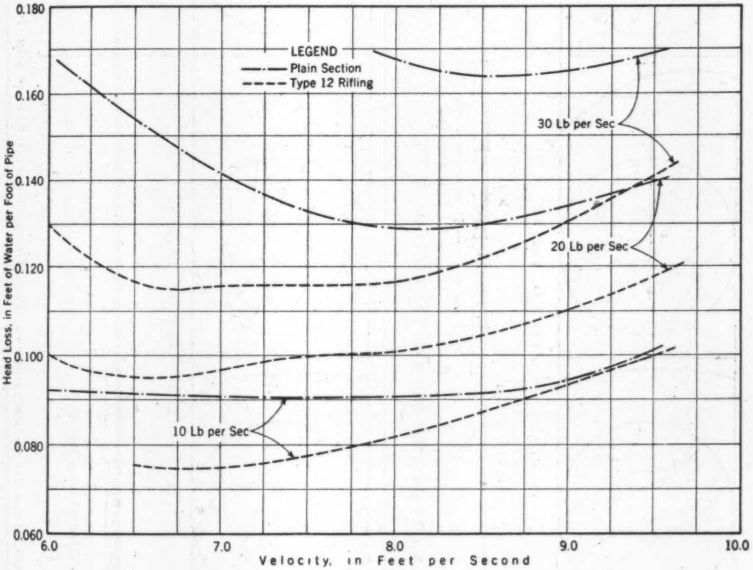
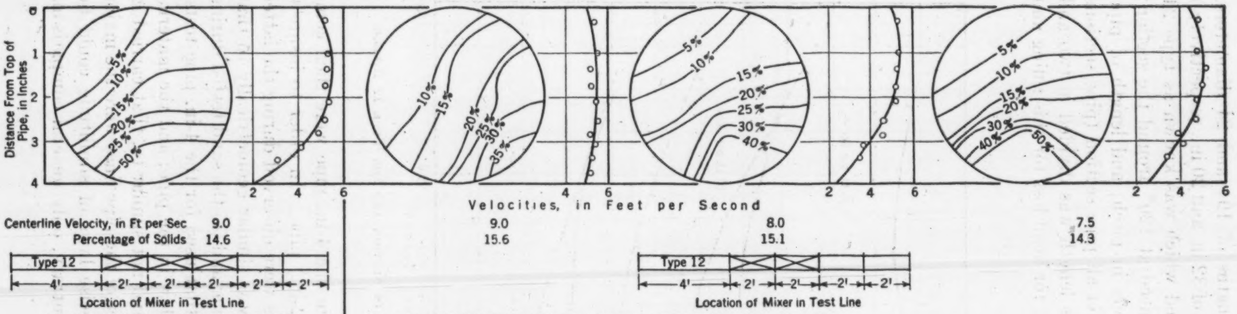


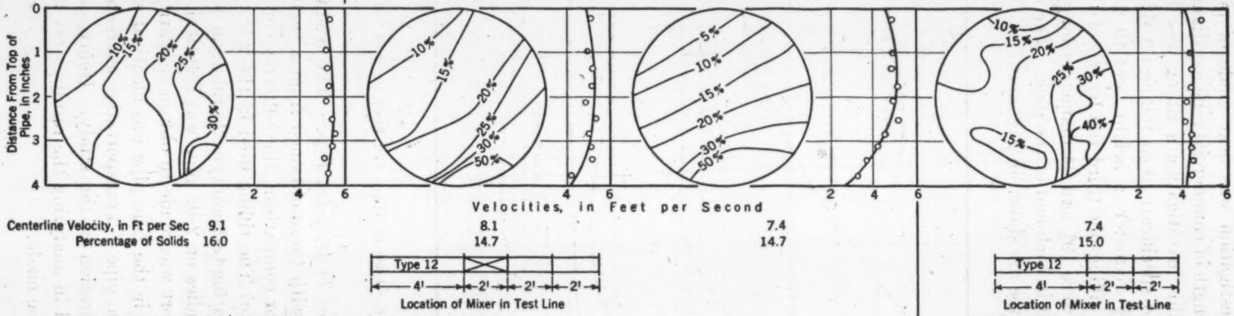
FIG. 6.—LOSS OF HEAD PER FOOT OF PIPE FOR A PLAIN SECTION AND FOR TYPE 12 RIFLING

Sand Tests, 2-In. Pipe.—The results from the 4-in. pipe were then compared with those obtained from a smaller (2-in.) pipe. In order to insure a basis for comparison, the following conditions were observed during the latter tests: (*a*) The rifling used in the 2-in. pipe was similar geometrically to that used during tests on type 24 rifling in 4-in. pipe; and (*b*) the sand tested during the studies on 2-in. pipe was identical with that used for the 4-in. pipe tests. No effort was made to obtain similarity of flow in the pipe; and the material moved in the 2-in. pipe was identical with, and not similar to, that carried in the 4-in. pipe. Transportation characteristics of the pipe lines were the major consideration of the study. Hence, no precise laws of similitude could be applied in analyzing the results. On the contrary, only general comparisons could be made.



(*a*) SOLID DISTRIBUTION WITH MIXER 10 FEET FROM END OF PIPE

(*b*) SOLID DISTRIBUTION WITH MIXER 8 FEET FROM END OF PIPE



(*c*) SOLID DISTRIBUTION WITH MIXER 6 FEET FROM END OF PIPE

(*d*) SOLID DISTRIBUTION WITH MIXER 4 FEET FROM END OF PIPE

FIG. 7.—SPACING TEST, TYPE 12; OBSERVATIONS OF SOLID DISTRIBUTION AT END OF 4-IN. PIPE

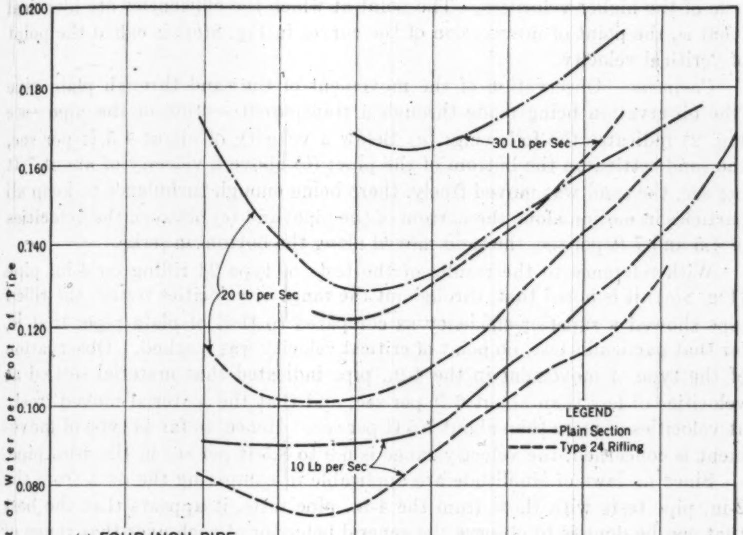


FIG. 8.—LOSS OF HEAD PER FOOT OF PIPE IN A PLAIN SECTION AND WITH TYPE 24 RIFLING

Fig. 8(*b*) presents, graphically, the results of the tests on 2-in. pipe. It is to be noted that the curves for the rifled and plain pipe intersect. At the lower velocities the rifled pipe has a superior efficiency, whereas the reverse is true of the higher velocities. The point at which the efficiencies are identical (that is, the point of intersection of the curves in Fig. 8(*b*)) is called the point of "critical velocity."

Comment.—Observation of the movement of the sand through plain pipe (the observation being made through a transparent section of the pipe—see Fig. 2) indicates the following: (*a*) Below a velocity of about 4.5 ft per sec, the sand settled on the bottom of the pipe; (*b*) above a velocity of about 7 ft per sec, the sand was moved freely, there being enough turbulence to keep all particles in motion along the bottom of the pipe; and (*c*) between the velocities of 4.5 and 7 ft per sec, the sand moved along the bottom in jerks.

With reference to the results of the tests of type 24 rifling on 4-in. pipe (Fig. 8(*a*)) it is noted that, throughout the range of velocities tested, the rifled pipe showed a superior efficiency as compared to that of plain pipe; that is, for that particular test, no point of critical velocity was reached. Observation of the type of movement in the 4-in. pipe indicated that material settled at velocities of less than about 6 ft per sec, and that the material moved freely at velocities of more than about 7.5 ft per sec. Hence, so far as type of movement is concerned, the velocity range is 6.0 to 8.5 ft per sec in the 4-in. pipe.

Since no laws of similitude are applicable in comparing the data from the 2-in. pipe tests with those from the 4-in. pipe tests, it appears that the best that can be done is to observe the general behavior of each over that range of velocities where similar types of movement occurred. Such general comparison indicates, in each case, a superior efficiency for rifled pipe up to the velocity at which material begins to move freely in the pipe. At higher velocities, the rifled 2-in. pipe loses its superiority and becomes less efficient than plain pipe. In the case of the larger pipe, however, the superior efficiency of the rifled section is maintained for velocities considerably in excess of the one at which material begins to move freely.

TESTS TO DETERMINE THE EFFECTS OF RIFLING ON PIPE CARRYING MATERIAL OTHER THAN SAND

After determining that rifling would produce beneficial effects when a sand of large-grain diameter was transported, the problem was resolved into a determination of the effects when other materials were being transported. Accordingly, tests were made using silt, clay, and pea gravel as the transported materials.

Silt.—The material used in this study was a loess obtained on the laboratory grounds, the mechanical analysis of this material being shown in Fig. 4. The silt was transported through the pipe in suspension, the completeness and evenness of which demonstrated that the distribution of solids was practically constant over the cross section. This phenomenon of transport in suspension in the case of silt is radically different from that in the case of sand; the latter is carried mostly in the lower part of the pipe line. The curves of Fig. 9(*a*)

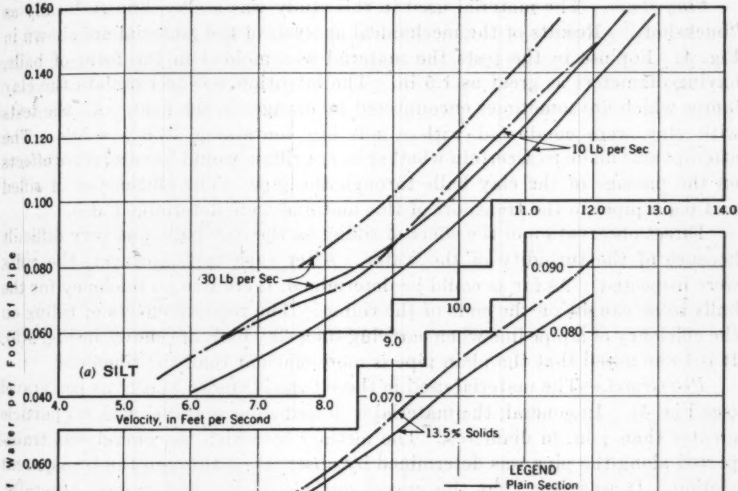


FIG. 9.—LOSS OF HEAD PER FOOT OF 4-IN. PIPE TRANSPORTING VARIOUS MATERIALS

are presented to show comparable characteristics of plain and rifled pipe carrying silt at various velocities and in identical quantities. They indicate superior efficiency for the plain pipe.

Clay Balls.—The material used in this study was a clay, known locally as "buckshot." Results of the mechanical analysis of the material are shown in Fig. 4. For use in the tests the material was molded in the form of balls, having diameters as great as 1.5 in. The intention was to simulate the clay lumps which are sometimes encountered by dredges in the field. All the tests with clay were conducted with a mixture containing 13.5% solids. The attempt was made to ascertain whether or not rifling would have adverse effects on the passage of the clay balls through the pipe. The efficiencies of rifled and plain pipes in the transport of this material were determined also.

Direct observation of the effect of rifling on the clay balls was very difficult because of the turbidity of the water. After each test, however, the rifles were inspected. As far as could be determined, there was no tendency for the balls to be caught on the ends of the rifling. The relative effects of rifling on the efficiency of a pipe line when carrying such clay balls are shown in Fig. 9(*b*). It is to be noted that the plain pipe is more efficient than the rifled one.

Pea Gravel.—The material used in these tests is known locally as pea gravel (see Fig. 4). In general, the material is described as a gravel with no particle greater than ¼ in. in diameter. The method by which the gravel was transported along the pipe was determined by observation through the transparent section. It was seen that the gravel settled quickly and moved generally, without jerking along the bottom of the pipe.

The curves of Fig. 9(*c*) are presented to show the effects of rifling on the efficiency of a pipe line carrying pea gravel. It is to be noted that throughout the range of velocities tested the rifled pipe line shows an efficiency higher than that of the plain pipe. It is to be noted also that the curves are similar to those already determined for pipe lines carrying sand (see Fig. 8(*a*)).

Comment.—The experiments indicate that rifling in a pipe line increases the efficiency of the pipe in transporting materials that are large and heavy enough to settle and travel along the bottom of the pipe (sand and gravel). This statement must be qualified as follows: As velocities and turbulence increase, any given material tends more and more to move in suspension rather than along the bottom. As this tendency increases, the relative efficiency of the rifling in the pipe line decreases. Finally, a point is reached where the resistance to flow offered by the rifling more than offsets the benefits derived from the mixing effects of the rifling. Beyond this point the rifled pipe is less efficient than the plain. This phenomenon is best illustrated by the data from the tests on 2-in. pipe carrying sand (Fig. 8(*b*)).

USE OF RIFLING ON HYDRAULIC DREDGES

In order to use rifling effectively on dredges on the Mississippi River, it is believed that the only satisfactory installation would be on channel-maintenance dredges which work in coarse sand and small gravel the greater part of their operating time. When material favorable for the use of rifles is being dredged, rifling a pipe line can be considered equivalent to installing a booster pump in the line. Thus, for long pipe lines or low-powered dredges, it would be possible to increase production without making mechanical installations on the dredge itself.

It is desired particularly to emphasize the point that the justification for rifling a discharge line is dependent upon the class of material encountered and the power of the pumping unit. There is a different problem on each dredge, therefore, and a careful estimate of the situation should be made in respect to the power of the dredge and the class of material to be encountered before it is decided to make any installations of rifling.

CONCLUSIONS

From consideration of the investigation described herein, the following conclusions are believed justified:

(1) Rifling will increase the efficiency of the line when the following materials are transported: (*a*) Coarse sand, and (*b*) gravel.

(2) Rifling will reduce the efficiency of the line when the following materials are transported: (*a*) Silt, and (*b*) clay.

For general use in considering the advisability of the use of rifling, the following is offered as a criterion:

> Rifling in the discharge line of a dredge will increase the efficiency of the line in cases where the material being dredged through a plain pipe would settle along the bottom in appreciable quantities.

ACKNOWLEDGMENTS

The investigations described herein were conducted at the U. S. Waterways Experiment Station, at Vicksburg, Miss. They were made under the supervision of the writer and under the direction of Francis H. Falkner, Captain, Corps of Engineers, U. S. Army. The writer wishes to thank Paul W. Thompson, Jun. Am. Soc. C. E., Captain, Corps of Engineers, U. S. Army (director of the station, 1937–1939), for his cooperation in making this paper possible and for suggestions in connection with its preparation.

DISCUSSION

Fred R. Brown,[4] Jun. Am. Soc. C. E. (by letter).—Although the subject of rifled pipe is not entirely new, little has been done to investigate or make known its possibilities. As stated by the author, care must be taken in deciding whether there is justification for the use of rifled pipe, and in the determination of the specifications for the rifles, if such justification is found. Too many variables are present to justify any definite conclusions as to the specifications for the best type of rifling. For example, combinations of height and length of rifle can vary and yet give identical results. The matter of the velocity of the transported material is another item that will influence the specifications for rifling in pipes.

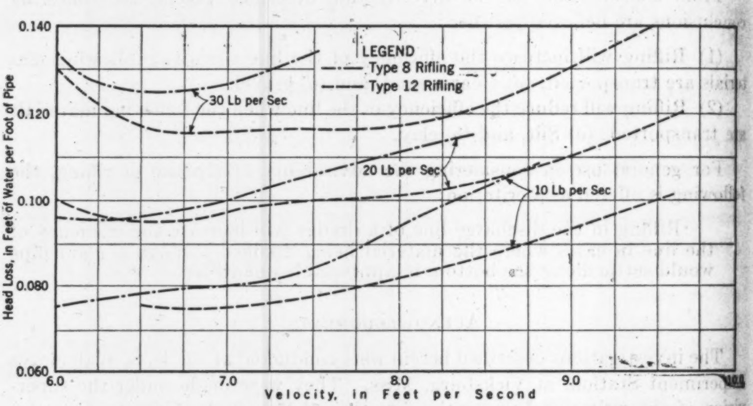


Fig. 10.—Loss of Head per Foot of Pipe for Type 8 and Type 12 Rifling

The type 12 mixer (see Table 1), developed as a result of the first series of tests discussed by the author, was the most satisfactory for pipes transporting coarse material. Type 8, however, indicated better mixing qualities. The only difference in the specifications of the two was in rifle height; type 8 was ¾ in. high and type 12 was ½ in. high. Decreasing the rifle height for equivalent lengths of rifling increases the efficiency of the pipe for concentrations of material above the critical zone in which the pipe may become blocked (see Fig. 10). For low velocities and high concentrations of material, however, the type 8 mixer in the pipe line permitted more material to be carried before blocking off than did the type 12 (see Fig. 11). Despite the fact that the type 8 mixer increased the mixing of the material, the great increase in overall efficiency, as a result of decreasing the rifle height, indicated that the type 12 mixer was the better design.

In order to determine the most effective spacing in the pipe line, the two mixers, types 8 and 12, were installed at various distances from the end of the pipe line. Solids distribution samples were taken for several velocities and used as a means for determining the most effective spacing for satisfactory operation. Investigations on plain pipe had shown a concentration of material in the lower part of the pipe and a marked decrease in velocity within the area of maximum solid concentration. In determining the proper spacing for mixers, the location desired was the position in the line just upstream from the point at which concentration on the bottom would occur. A fairly uniform velocity distribution would occur in this location, thus utilizing all effects of the spiral motion. If the spacing of the mixers were too great, the material would all be concentrated on the bottom of the pipe and would require an amount of energy comparable to that originally expended to impart a spiral motion to the flow once more. The results of the spacing tests on the type 8 mixer were comparable with those shown in Fig. 7. As in the tests on the type 12 mixer presented by the author, a spacing of 20 diameters was indicated in order to obtain satisfactory mixing at the maximum distance between sections. Comparison of the spacing tests indicated that with a spacing of 20 diameters there was a better mixing of material through use of the type 8 mixer. However, the mixing was not sufficiently increased to justify the ½-diameter reduction in clear pipe diameter between types 8 and 12. Furthermore, in the light of the head-loss comparison discussed previously, it seems justifiable to state that type 12 is the better of the two designs.

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[4] Junior Engr., U. S. Waterways Experiment Station, Vicksburg, Miss.

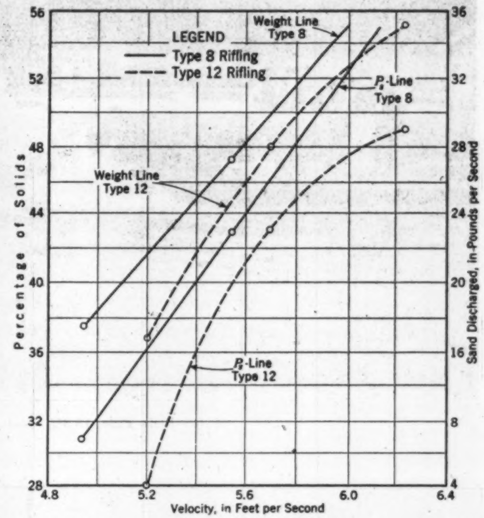

FIG. 11.—RELATION BETWEEN "BLOCKING" POINTS FOR TYPE 8 AND TYPE 12 MIXERS

The type 12 mixer, which the author states was superior to the other types tested during the first series, was selected for field tests. The dredge was located on the Mississippi River, north of Memphis, Tenn., in a selected reach of the river to insure the pumping of coarse materials. The results of these tests indicated that the type 12 mixer was superior to plain pipe. Fig. 12 shows the discharge from the end of the pipe for plain pipe and for rifled pipe. It is to be noted that the material is more thoroughly mixed when discharging from the rifled test line (Fig. 12(*b*)) as is evidenced by the decrease in spray at the end of the pipe. H. S. Gladfelter,[5] M. Am. Soc. C. E., has shown the distribution of solids across the test pipes for the conditions shown in Fig. 12.

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[5] *Transactions*, Am. Soc. C. E., Vol. 104 (1939), p. 1374.

(*a*) Plain Pipe; 13.3% Solids

(*b*) Type 12 Rifled Discharge Pipe; 18.1% Solids

FIG. 12.—DISCHARGE FROM A 32-IN. DREDGE; PUMP SPEED 150 RPM

The sections presented by Mr. Gladfelter indicate, definitely, the mixing ability of rifled pipe. Results of these preliminary field tests led to the further development of a mixer more practicable for field installation. Additional mixers investigated are described in Table 1(*b*). The writer hopes that the author will enlarge on the results of the aforementioned field tests, thus advancing, another step, the knowledge of effect of rifling in pipe lines.

David L. Neuman,[6] M. Am. Soc. C. E. (by letter).—In his "Conclusions," Mr. Howard states a criterion for determining the advisability of the use of rifling as follows: "Rifling in the discharge line of a dredge will increase the efficiency of the line in cases where the material being dredged through a plain pipe would settle along the bottom in appreciable quantities." Such a condition of insufficient discharge velocity to keep the materials in suspension is generally due to insufficient power on the main pump for the length of discharge line then in use. A booster pump may be placed in the discharge line or, to meet immediate and anticipated needs, the power unit may be replaced by one of higher power. Generally, replacement of the pump is required at the same time.

Repowering a dredge is an expensive and time-consuming process. If the length of discharge line, at the time the insufficient discharge velocities become apparent, is about the maximum length to be used, and if the dredge is to work in silt or clay only a small part of its total working time, an alternate (presented by Mr. Howard) of using a rifled discharge line becomes attractive.

The 20-in. pipe line cutter dredge, *Henry Bacon*, operating under the Wilmington, N. C., Engineer District, was used on maintainance work along the coastal waterways of Georgia and the Carolinas in 1936. Most of the work was in sand, or materials containing a large proportion of sand. With a maximum of 650 indicated horsepower available for the main pump, it was underpowered for the work assigned. To determine whether the use of a rifled discharge line would be of material assistance, tests were made in January, 1937, on the Savannah River, about 10 miles below Savannah, Ga.

The rifled discharge line was equipped with the laboratory type 24 rifling described by Mr. Howard. Each 40-ft length (24 diameters) of 20-in. pipe had a set of three rifles at each end, spaced at 120° around the section, of 6.83 ft axial length. The rifles were of $\frac{5}{16}$-in. plate, 2½ in. high, having a pitch of 16.67 ft, spot-welded into position. The discharge line, totaling 907 ft in length, was made up of 120 ft of dredge hull piping and 787 ft of exterior floating line. Distributed in the hull piping were four sets of rifling totaling 36 ft in length. A special gage was attached to the hull piping to obtain samples of the material and an indication of the percentage of solids passing through the line. At 15-min intervals discharge velocities were determined by a method (similar to the electro-saline method) which will be referred to as the electro-acid method. Use of concentrated sulfuric acid in lieu of a saline solution was made necessary by the presence of sea water at the site of the work. Steam and fuel consumption, as well as the power output of the main pump engine, was measured for each run. Bottom surveys were made before and after each run to obtain dredge outputs.

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[6] Lt. Col., Corps of Engrs., U. S. Army, Fort Belvoir, Va.

The test had to be made during normal maintainance operations and without undue interference with the operation schedule. The intensions had been to make the tests at a place on the inland waterway north of Savannah where only sand would be encountered. Because of a delay in completing preparations for the test, they had to be run at a site on the Savannah River where materials were encountered varying from sand to a stiff combination of sand, silt, and clay, which was difficult to cut and dredge. When operating in the tough, clayey material, the power available on the cutter was too low to feed material to the suction mouth at a rate sufficiently high to load the discharge line fully.

Because of the variation in the materials encountered, the test as a whole was unsatisfactory, in that the relative effectiveness of rifled discharge piping and plain discharge piping was not established. The only justifiable direct comparison, in similar material (sand) and operating at the same pump speed (150 rpm), can be made for one test with plain pipe and one with rifled pipe. The average output with plain pipe was 860 cu yd per hr, place measurement, and with rifled pipe, 1,134 cu yd per hr, place measurement, which denotes an increase of output of rifled pipe over plain pipe of 32%.

No plugging of the discharge line occurred while using either the plain or rifled pipe. In one instance, while operating in clay, the suction line became completely plugged for a distance of about 8 ft immediately in the rear of the suction mouth. It is believed that a suitably designed single rifle placed in the suction line would have prevented the plugging of the suction line. The rifled discharge line was able to pass large balls of clay. No large roots or stumps were encountered. A few weeks after these tests the dredge, with a rifled discharge line, was operated in an area where the material (mud and silt) contained an unusually large number of sawmill slabs and roots. Plugs of the discharge line did occur but, in the opinion of the dredge operators, no more than would have occurred if plain discharge line had been used.

The results obtained in the aforementioned field test of rifled pipe line fully support Mr. Howard in his conclusions and criterion. Had the encountered material been only sand and coarser uncemented materials the value of rifling to the *Henry Bacon* for obtaining appreciably increased outputs when working in such materials would have been evident.

Acknowledgment.—The data used in this discussion were obtained from the report of the District Engineer at the U. S. Engineer Office in Wilmington. The test was the result of the cooperation of the U. S. Waterways Experiment Station, the Memphis Engineer District, and the Wilmington District. Mr. Howard was present at the test, as representative of the U. S. Waterway Experiments Station.

R. Y. NEWELL, JR.,[7] ESQ. (by letter).—In general the writer agrees with the conclusions reached by the author, and in this connection it is interesting to compare results in the laboratory using the 4-in. test pipe with those obtained during actual operation of a dredge having a 32-in. discharge pipe line. Reference is made to preliminary field tests of rifled discharge pipe made during June, 1936, with the dredge *Jadwin*, a 32-in. pipe-line, dust-pan type, hydraulic dredge of the U. S. Engineer District, Memphis, Tenn.

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[7] Marine Engr., U. S. Maritime Comm., Washington, D. C.

The location, at a point in the Mississippi River a few miles upstream from Memphis, was selected for making the test because of the coarse and heavy characteristics of the sand forming an obstructing bar at that point. A sieve analysis of a representative sample is shown in Table 2.

TABLE 2.—MECHANICAL ANALYSIS OF REPRESENTATIVE SAMPLE OF MATERIAL DREDGED DURING TESTS ON THE DREDGE *Jadwin*

FINE SAND			MEDIUM SAND			COARSE SAND			FINE GRAVEL			MEDIUM GRAVEL		
Grain Size, in Millimeters		Percentage of total	Grain Size, in Millimeters		Percentage of total	Grain Size, in Millimeters		Percentage of total	Grain Size, in Millimeters		Percentage of total	Grain Size, in Millimeters		Percentage of total
From	To		From	To		From	To		From	To		From	To	
0.10	0.25	10.0	0.25	0.50	33.0	0.50	1.00	40.0	1.00	2.00	10.0	2.00	10.00	7.0

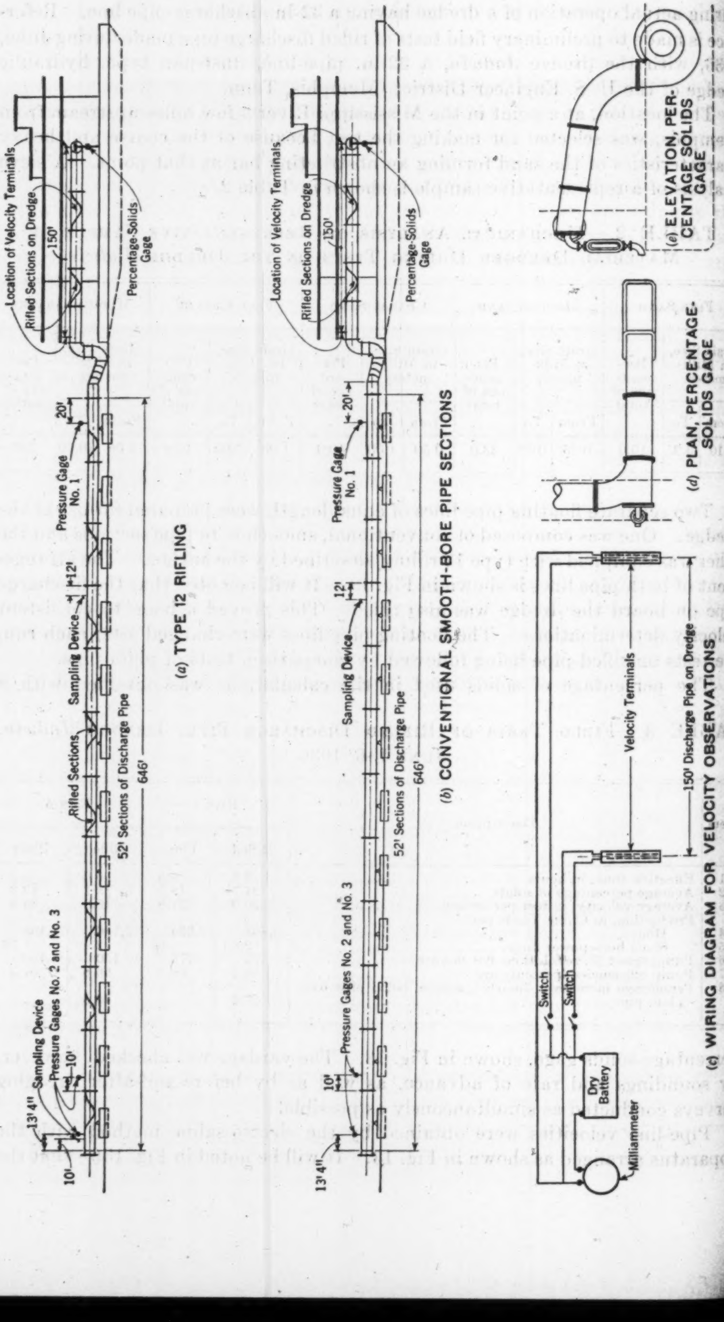
Two separate floating pipe lines of equal length were prepared for use of the dredge. One was composed of conventional, smooth bore pipe sections and the other was equipped with type 12 rifling, described by the author. The arrangement of both pipe lines is shown in Fig. 13. It will be noted that the discharge pipe on board the dredge was also rifled. This proved a boon to consistent velocity determinations. The floating pipe lines were changed after each run, the tests on rifled pipe being followed by comparison tests of plain pipe.

The percentage of solids used in the calculations was obtained with a percentage-solids gage, shown in Fig. 13. The yardage was checked, however, by soundings and rate of advance, as well as by before-and-after dredging surveys conducted as simultaneously as possible.

TABLE 3.—FIELD TESTS OF RIFLED DISCHARGE PIPE, DREDGE *Jadwin*, JUNE 1–6, 1936

Item	Description	RUN 4		RUN 5	
		Rifled	Plain	Rifled	Plain
1	Effective time, in hours	5.5	5.5	5.0	5.0
2	Average percentage of solids	24.7	18.2	18.1	13.3
3	Average velocity, in feet per second	20.3	21.8	15.8	14.8
	Production, in Cubic Yards per:				
4	Hour	3,808	2,994	2,130	1,464
5	Shaft horsepower hour	2.63	1.97	2.42	1.76
6	Pump speed, in revolutions per minute	175	175	150	150
7	Pump efficiencies (percentages)	68.5	66.8	66.8	67.2
8	Percentage increase in hourly yardage, rifled pipe over plain pipe	27.2	....	45.5	....

Pipe-line velocities were obtained by the electro-saline method with the apparatus arranged as shown in Fig. 13. It will be noted in Fig. 13(*c*) that the



(a) TYPE 12 RIFLING

(b) CONVENTIONAL SMOOTH-BORE PIPE SECTIONS

(c) WIRING DIAGRAM FOR VELOCITY OBSERVATIONS

(d) PLAN, PERCENTAGE-SOLIDS GAGE

(e) ELEVATION, PERCENTAGE-SOLIDS GAGE

Location of Velocity Terminals
Rifled Sections on Dredge
150'
Percentage-Solids Gage
20'
Pressure Gage No. 1
12'
Sampling Device
Rifled Sections
52' Sections of Discharge Pipe
646'
Sampling Device
Pressure Gages No. 2 and No. 3
10'
13' 4"
10'
Velocity Terminals
150' Discharge Pipe on Dredge
Switch
Switch
Dry Battery
Milliammeter

FIG. 13.—PLANT LAYOUT, COMPARATIVE OBSERVATIONS ON RIFLED AND PLAIN PIPE

terminals were separated by rifled pipe. This fact was responsible for consistent and reasonable results, even when high concentrations of solids were being transported, whereas the method gives highly erroneous results under similar conditions when the terminals are separated by smooth pipe. Data on the distribution of solids across the plain and rifled pipe during these tests have been presented by H. S. Gladfelter,[8] M. Am. Soc. C. E.

Every effort was made throughout the tests to obtain the maximum output of the dredge under all conditions. The results of the tests are shown in Table 3, and it will be noted that the greater advantage of rifled pipe over plain pipe is evidenced at the reduced pump speed. This indicated that rifling would be advantageous when used on dredges of low power or to compensate for low pipe-line velocities due to any number of causes. The yardage data (item 4, Table 3) are based on computations made from the percentage-of-solids readings and the discharge velocity.

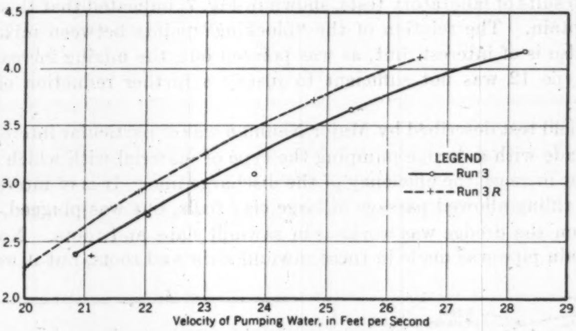


Fig. 14.—Loss of Head in Plain and Rifled Pipe

Fig. 14 shows the comparison of friction losses in plain and rifled pipe lines when pumping water, the loss for the rifled pipe being higher throughout the range. It appears, however, that the situation is reversed when heavy sands or gravel are being transported through the pipes, due to the mixing effect of the rifles.

Although the rifled pipe showed definite advantage during the test described herein, the writer conducted subsequent tests with dredges pumping fine sand and silt in which the plain pipe was equally as efficient as the rifled pipe.

It appears from the foregoing that the conclusions drawn by Mr. Howard from the results of laboratory tests are substantially borne out by tests of dredges made under ordinary operating conditions.

G. W. Howard,[9] **Jun. Am. Soc. C. E.** (by letter).—The success or failure of any development in the laboratory is determined by trials in practice, and in discussing the field tests of rifled pipe it is desired to mention again the conclusions that were drawn from the laboratory studies. These are:

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[8] *Transactions*, Am. Soc. C. E., Vol. 104 (1939), p. 1374.

[9] Capt., Corps of Engrs., U. S. Army, Fort Belvoir, Va.

> "(1) Rifling will increase the efficiency of the line when the following materials are transported: (*a*) Coarse sand, and (*b*) gravel.
>
> "(2) Rifling will reduce the efficiency of the line when the following materials are transported: (*a*) Silt, and (*b*) clay."

As shown in the photographs presented by Mr. Brown (Fig. 12), the material in field tests was more evenly distributed in the rifled pipe than in the plain pipe. Results of laboratory tests, shown in Fig. 7, indicated that this condition would obtain. The relation of the "blocking" points between mixer types 8 and 12 also is of interest, but, as was pointed out, the mixing increase of type 8 over type 12 was not sufficient to justify a further reduction of the pipe diameter.

The field test described by Major Neuman was of particular interest because it was made with a dredge pumping the type of material with which the rifling would not increase the efficiency of the discharge line. It is of interest to note that the rifling allowed passage of large clay balls, but was plugged, at a later date, when the dredge was working in sawmill slabs and roots. No comparison of plain pipe was made in these sawmill slabs and roots, but it was pointed out by Major Neuman that, in the opinion of the dredge operator, there were no more plugs than would have occurred if a plain discharge line had been used.

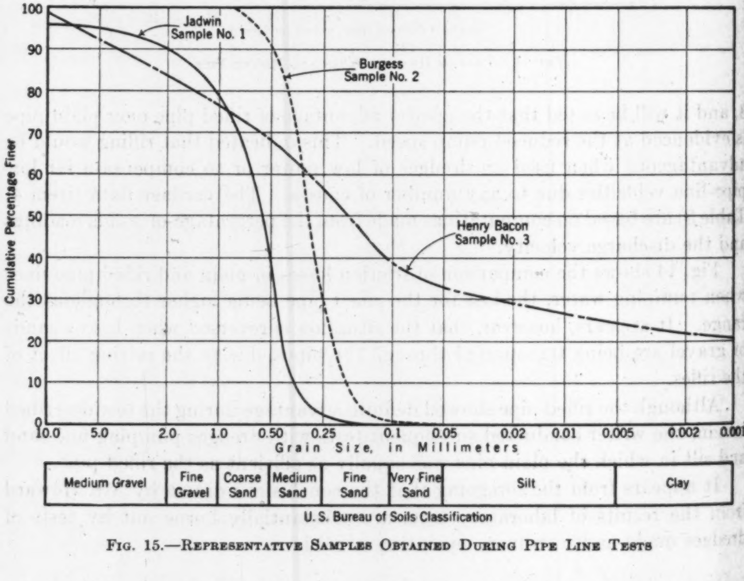

FIG. 15.—REPRESENTATIVE SAMPLES OBTAINED DURING PIPE LINE TESTS

The data presented by Mr. Newell include basic information on a complete field test. It should be noted, however, that the conditions under which the pipe was tested were the most favorable of all that were encountered in the various field tests. Later tests by the Memphis Engineer District using the dredge *Burgess* did not show favorable results, although it was not shown that rifling reduced the efficiency of the line. These tests were made in a much finer material. Fig. 15 shows the mechanical analyses of material pumped in the tests on the dredges *Jadwin* (Mississippi River, Cedar Point Bar), *Burgess* (Mississippi River, Mile 230), and *Henry Bacon* (Savannah River). It can be seen that only the dredge *Jadwin* was working in material classified as coarse sand.

Short tests, such as the three mentioned, each of which lasted about one week, cannot be considered as conclusive. They do indicate, however, that rifling is of value under certain field conditions.

A test which lasted for two years was completed by the Memphis Engineer District.[10] This test consisted in equipping one dredge with rifling and comparing its output record with that of another dredge of identical construction. Chief conclusions from this study of production data were:

(1) Increase of production (14.7%) for dredge with rifled line;
(2) Decrease of fuel costs (9.5%) for dredge with rifled line; and
(3) Decrease of total costs ($0.0038 per cu yd) for dredge with rifled line.

The elapsed time of this test is such that the data should be considered of more significance than for the other studies. The fact that the test covered all operating conditions encountered by the dredges during two years also is of significance.

In closing it is desired again to emphasize the fact that rifling the discharge line of hydraulic dredges is not a "cure-all" for output problems. Laboratory tests, confirmed qualitatively by results of field tests, however, have indicated the validity of the following criterion:

> Rifling in the discharge line of a dredge will increase the efficiency of the line in cases where the material being dredged through a plain pipe would settle along the bottom in appreciable quantities.

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[10] "Rifling in Dredge Pipe Lines," *The Experiment Station Bulletin*, Vol. 2, No. 1, February 1, 1939, pp. 1–5.

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PROBLEMS AND TRENDS IN ACTIVATED SLUDGE PRACTICE

BY ROBERT T. REGESTER,[1] M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. LANGDON PEARSE, FRANK C. ROE, GERARD A. ROHLICH AND CLAIR N. SAWYER, E. SHERMAN CHASE, AND ROBERT T. REGESTER.

SYNOPSIS

The activated sludge process is used to an increasing extent by cities in which local conditions require a high degree of sewage treatment. Problems presented in the design and operation of activated sludge plants in the United States, and trends indicative of American practice since 1930, are discussed in this paper. Pertinent data relating to the basic design and principal characteristics of thirty plants, of large and medium capacities, are included for comparative study. Available operation results for certain works are given. In conclusion, the need for further research is noted.

INTRODUCTION

The extensive use of the activated sludge process for municipal sewage treatment is a tribute to the efforts of early investigators in this field and to engineers who were responsible for its adoption on the first large-scale projects. Early experiments (1915 to 1921) demonstrated the high efficiency of the process in comparison with certain other methods and furnished basic information for the design of the large works at Milwaukee, Wis., Chicago, Ill., and Indianapolis, Ind. The operating results and experiences obtained from these and other works since 1925, together with further experimentation, have largely guided the design of subsequent works.

Advances in design and improvements in the methods of operation, particularly since 1930, have marked the endeavors to solve numerous pertinent problems. The general objectives have been: (*a*) More consistent performance under variable conditions; (*b*) greater economies in both construction and operation; (*c*) better technique for process control; and (*d*) increased facility of operation.

NOTE.—Published in November, 1939, *Proceedings*.

[1] Cons. Engr., Baltimore, Md.

ACTIVATED SLUDGE PLANTS

The total nominal capacity of activated sludge plants in operation (1916 to 1938, inclusive) emphasizes the increased use of the process in the United States (see Fig. 1). The number of these plants exceeded two hundred in 1938. In 1939, with large works at Chicago (Southwest), and New York, N. Y. (Tallmans Island), included, the total nominal capacity in operation is estimated as 1,700 mgd.

The rapid increase of the total nominal capacity in operation during the period 1934 to 1938 reflects to some degree the stimulation from federal funds, although certain large projects constructed in this period were contemplated prior to federal participation. It is noteworthy that litigation involving the infringement of the process patents, extending from 1924 until the expiration of the last adjudicated patent on May 25, 1937, did not materially retard the adoption of the process for new plants.

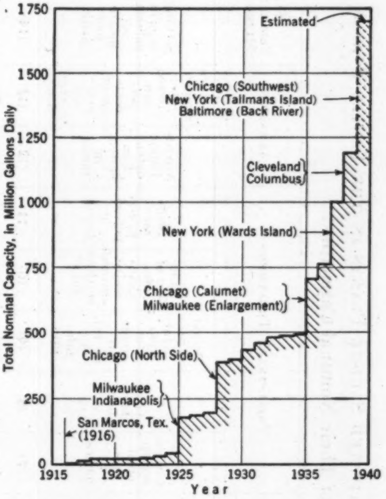

FIG. 1.—INCREASED USE OF ACTIVATED SLUDGE PROCESS OF SEWAGE TREATMENT IN THE UNITED STATES

Design data relating to thirty activated sludge plants of large and medium capacities are included in Table 1. Their total nominal capacity is approximately 1,600 mgd, and that of the ten largest works is nearly 1,400 mgd. Partial-aeration works, in series with trickling filters and other secondary treatment, at Cleveland, Ohio (Southerly), Decatur, Ill., Fort Worth, Tex., and elsewhere, are omitted from Table 1 so as to avoid possible misunderstanding of the data.

General Design.—With few exceptions, activated sludge plants of large and medium capacities, constructed since 1930, have included coarse screens, grit chambers, and settling tanks as preliminary treatment devices. These have been supplemented in some plants by pre-aeration chambers for grease separation. The devices for principal treatment have included the usual aeration tanks and final settling tanks. In a few instances, provision also was made for sludge re-aeration. Further treatment in the form of chlorination, filtration, and cascade or weir aeration has been provided in certain works. The maximum capacity for which the principal treatment devices are designed has shown a tendency to increase from 150% to 200% of the average sewage flow.

Based upon the experience at Chicago (North Side, Item 3, Table 1) of treating, satisfactorily, a sewage flow considerably in excess of the plant's design capacity, L. C. Whittemore, M. Am. Soc. C. E., has reported (1)[3] that the

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[3] Numerals in parentheses refer to corresponding references in the Appendix.

TABLE 1.—DESIGN DATA FOR THIRTY ACTIVATED SLUDGE PLANTS IN THE UNITED STATES
(Arranged in the Order of Their Nominal Capacities)

Item No.	Location	Nominal Capacity		Year of Starting		Type of sewerage system[i]	Preliminary Treatment					Aeration		Final Sedimentation, Mixed Liquor		Further treatment[ee]
		Sewage flow, in million gallons daily	Human population, in thousands	Construction	Operation		Coarse screen openings, in inches	Grit chambers	Period of preaeration for grease removal, in minutes	Number of mechanically cleaned settling tanks[p]	Sedimentation period, in hours	Mixed-liquor detention period, in hours	Return sludge, expressed as a percentage of average sewage flow[x]	Settling rate, in gallons per square foot daily	Detention period, in hours	
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
1	Chicago, Ill. (Southwest)	400	1,300[d]	1935	1939	C[j]	2	No	11.0[b]	12	0.57	5.0	20	1,200	1.7	C[dd,ee]
2	New York, N. Y. (Wards Island)	180	1,230	1931	1937	C	1	Yes	No	8[q]	1.0	5.8	20	890	2.6	No
3	Chicago, Ill. (North Side)	175	800	1923	1928	C	1	Yes	No	8[q]	0.5	6.3	20	1,180	1.9	No
4	Milwaukee, Wis.	155	...[g]	1920 1934[f]	1925 1935[f]	C	2	Yes	No	0[r]	...	5.7 and 6.0[f]	25	1,030[s] 1,080[f,aa]	2.3[f]	No
5	Chicago, Ill. (Calumet)	136	455	1931	1935	C	5/8	No	No	4	0.17	5.0	20[x]	1,200	1.8	No
6	Cleveland, Ohio (Easterly)	123	770	1931	1938	C	Yes[l]	Yes[n]	6.0	8[s]	1.0	5.0	25	980	2.2	C
7	Indianapolis, Ind.	75[b]	...	1921[g]	1925 1937	C[j]	1	Yes	No	4[t]	...	5.5[b]	30[b]	1,400[b]	...	No
8	Columbus, Ohio	50	400	1934	1938	C	3/4	Yes	6.25	4[s]	1.5	6.0	30[v]	890	2.5	C[ff]
9	Baltimore, Md. (Back River)	40	...	1938[h]	...	S	3/4	Yes[o]	No	5[q,u]	...	5.0	25	1,000	2.4	No
10	New York, N. Y. (Talimans Island)	40	290	1937	1939	C	1	Yes	No	3	1.0	3.5	20	950	2.2	No
11	Providence, R. I.	36	...	1930	1936	C	3/4	Yes[o]	No	3[q]	1.5	5.0	25[x]	870	2.3	No
12	San Antonio, Tex.	30	400	1929	1930	S	1/2	Yes	Yes	2[q]	0.5	5.0	20[x]	1,410	1.9	No
13	Peoria, Ill.	22[e]	140	1929	1931	C	1 1/2	Yes	No	4[q]	1.5	10.5[w]	20	640[w]	4.2[w]	No
14	Houston, Tex. (North Side)	15[b]	...	1916 1929	1917	C	...	Yes	No	0	...	...	...[x]	...	...	No
15	Austin, Tex.	12	...	1936	1937	C	Yes	Yes[o]	...	1[u]	0.8	3.6	20	1,630	1.33	No
16	Phoenix, Ariz.	12	...	...	1932	S	1	No	No	1[q]	1.0	...	25[x]	770	2.7	C
17	Jackson, Mich.	11.25	...	1934	1937	C	Yes[m]	Yes	No	3	1.0	4.5	25	910	2.0	No
18	Pasadena, Calif.	11[b]	160[b]	1922	1924	S	13/16[m]	Yes	Yes	0[r]	...	5.5[b]	25[x]	700[b]	...	C

TABLE 1.—(*Continued*)

Item No.	Location	NOMINAL CAPACITY		YEAR OF STARTING		Type of sewerage system[i]	PRELIMINARY TREATMENT					AERATION		FINAL SEDIMENTATION, MIXED LIQUOR		Further treatment[cc]
		Sewage flow, in million gallons daily	Human population, in thousands	Construction	Operation		Coarse screen openings, in inches	Grit chambers	Period of preaeration for grease removal, in minutes	Number of mechanically cleaned settling tanks[p]	Sedimentation period, in hours	Mixed-liquor detention period, in hours	Return sludge, expressed as a percentage of average sewage flow[x]	Settling rate, in gallons per square foot daily	Detention period, in hours	
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
19	Lansing, Mich.[a]	9	100	1937	...	C	Yes[m]	Yes	10.0	4[s]	1.0	6.0	25	1,120	1.5	No
20	Springfield, Ill.	7.5	75	1927	1928	C	¾, 1	Yes	8.0	4[u]	1.0	7.7	20	1,000	2.3	No
21	Lima, Ohio	7	70	1930	1932	C	¾	Yes[o]	No	2[q]	1.6	4.5	25	890	2.0	No
22	Greensboro, N. C.	6.5	40[e]	...	1938	C[k]	1[m]	Yes[o]	No	4	...	6.0	25[b]	920[b]	1.8[b]	C
23	Madison, Wis.	6.25	...	1934	1936	S	Yes	Yes	3.0	2	1.5	5.5	25	1,000	2.16	A
24	Charlotte, N. C. (Irwin Creek)	6	...	1927	1928	...[k]	⅜	No	No	0	...	5.0	...	...	1.5[bb]	No
25	Durham, N. C. (North Side)	6	...	1934	1935	S[k]	Yes[m]	No	No	6	...	5.0	...	...	...	No
26	Lancaster, Pa. (North)	6	29	1933	1934	C	¾	Yes	3.0	2	1.6	6.3	20	1,200	1.8	C
27	Lancaster, Pa. (South)	6	55	1933	1935	C[j]	¾	Yes	3.0	2	1.6	6.3	20	1,200	1.8	C
28	Topeka, Kans. (East Side)	6	...	1935	1937	C[j]	1	Yes[o]	19.5	2[q]	2.7	6.0	20[x]	820	2.4	No
29	Charlotte, N. C. (Sugar Creek)	5	...	1927	1928	...	2	Yes	No	1[q,r]	2.5	5.0	...	...	1.5[bb]	No
30	Ann Arbor, Mich.	4.5	48	...	1937	S	Yes[m]	No	12.0	2	1.0	6.0	25	880	2.25	No

[a] Garbage to be ground and digested. [b] Approximate. [c] Includes 8-mgd beer slop and strawboard wastes. [d] Plus 1,000,000 industrial equivalent. [e] Plus 20,000 industrial equivalent. [f] 70-mgd enlargement. [g] New units built in 1936. [h] Two each; aeration and final tanks not yet under construction. [i] C = combined; S = separate systems. [j] Packing house wastes in sewage. [k] Textile wastes in sewage. [l] Comminutors plus original 1½- and ¾-in. racks. [m] Comminutors. [n] Detritors plus original units. [o] Detritors. [p] Rectangular in shape, with the exceptions given. [q] Square. [r] Fine screens; are $\frac{1}{12}$ in. at Item 4. [s] Covered or housed. [t] Treat concentrate from fine screens. [u] Circular. [v] 3 rectangular and 1 square. [w] Based on 11-mgd sewage flow. [x] Sludge is re-aerated only at Items 11, 12, 14, 16, 18, and 28; provision for re-aeration has been made at Item 5. [y] One tank equipped for experimental use. [z] Maximum, 2,000 gal per sq ft daily. [aa] Maximum, 1,600 gal per sq ft daily. [bb] Sewage only. [cc] C = chlorination; A = cascade aeration. [dd] Partial flow used for maintenance of condensers. [ee] Part chlorination and part cascade aeration. [ff] Future chlorination.

hydraulic design for the channels and conduits in the Southwest works provide for a possible 25% increase in the designed maximum capacity of the tanks. This provision anticipates a possible reduction of the aeration period.

The general arrangement of the large plants in particular is characterized by tanks of greater unit sewage capacity—20 to 25 mgd in the case of aeration tanks, and 10 to 12.5 mgd for final settling tanks. This trend reflects a desire for construction economy. Compactness and flexibility are general features that have received careful consideration.

In some plants equipped with raw sewage pumps, these pumps are housed in the same building with the air blowers and auxiliary equipment and, occasionally, sludge de-watering equipment with incinerators is also included.

By-passing arrangements are often provided for the principal treatment devices. In the works at Columbus, Ohio (2), provision is made for pumping final effluent, during occasional periods of high water in the Scioto River. Under these conditions, it was intended that a portion of the sewage flow would receive principal treatment, while the remaining portion would be by-passed directly to the river after receiving preliminary treatment only.

Service tunnels containing miscellaneous piping and electrical conduits are a special feature of several works. These tunnels, by connecting the various structures, permit easy access for the maintenance of essential services and also provide sheltered communication.

Provisions for future extensions of plants allow for either the construction of additional similar units to be connected directly to the existing ones in accordance with the original plan, or for units of a different design to be located in unassigned adjacent space. In view of past changes in both processes and equipment, the latter method has merit.

The effort to obtain greater operating economies has influenced general design. The use of power derived from by-products has been an important factor in the selection of sludge disposal methods. A notable example is the use of waste heat from the incineration of flash-dried sludge, combined with the burning of coal, for the generation of steam to operate turbine-driven pumps, blowers, and generators at the Chicago (Southwest) plant (3). In contrast, the pumps and blowers in the New York (Tallmans Island) works are driven entirely by engines that utilize gas to be obtained from the digestion of sludge. This latter method is in use at Topeka, Kans. (East Side) (4*a*). In both cases, provision is made for the use of purchased gas when needed. Other plants use sludge gas in conjunction with purchased electric power for operating air blowers.

The decision to treat certain industrial wastes together with domestic sewage has been recognized as an important design consideration. In older plants (5), this was evidenced by relatively long aeration periods. At Greensboro, N. C. (6), where textile wastes are treated with the sewage, flexibility has been provided to permit the use of either the activated sludge process or other methods as may be found most suitable.

PRELIMINARY TREATMENT

Screening and Grit Removal.—Mechanically cleaned bar screens, or comminutors, are installed in most plants. The usual range in the width of clear openings for the screens is from $\frac{5}{8}$ in. to 1 in. Screenings are generally macerated and returned to the sewage. Usually, mechanically cleaned chambers are used for the removal of grit from combined sewage. An interesting exception is the omission of grit chambers at the Calumet plant and the omission of both bar screens and grit chambers at the Southwest plant in Chicago. In commenting (3) on this change, Mr. Whittemore and Norval E. Anderson, M. Am. Soc. C. E., have stated that these structures are costly to build and operate, and with the adoption of sludge incineration they believe that screens and grit chambers may not be necessary. However, the plants are designed to permit the future installation of grit chambers. Notes on the operation of the Calumet works (7) during 1937 indicate that no difficulty has been experienced in the removal of grit with primary sludge at times of normal sewage flow. On the other hand, periods of storm have shown the need for standby facilities to remove excess grit from the sludge.

Grease Separation and Removal.—A greater appreciation of the benefits to be obtained from the pre-aeration of screened sewage for the separation of grease is evidenced by such provision in a number of plants with usual detention periods of 3 to 12 min. Grease is removed mainly with skimming devices in the preliminary settling tanks. The inhibiting effect of grease film on the rate of oxygen absorption by sludge floc stresses the importance of this step in pretreatment.

Preliminary Sedimentation.—It is interesting to note that, in nearly all of the thirty plants listed in Table 1, one or more preliminary settling tanks were provided, except where fine screens were installed. Doubts have been expressed as to the necessity for these tanks, and in at least one plant the tanks were by-passed for a period without apparent difficulty.

The principal reasons for including preliminary sedimentation in the Cleveland (Easterly) plant have been stated by George B. Gascoigne, M. Am. Soc. C. E., as follows (8): (1) To provide a sewage for aeration having uniform characteristics in that the effect of concentrated industrial wastes and of pollution from the first flush of storms would be minimized; (2) To provide optimum conditions for the rapid digestion of the deposited solids obtained from the excess activated sludge; (3) To reduce the volume of sludge that must be handled and disposed of; (4) To reduce the quantity of air required; and (5) To provide partial treatment for the first flush from storms.

Odor Control.—The location of activated sludge plants in close proximity to residential areas has necessitated means for the control of odors emanating from preliminary treatment devices. To this end, tanks at Cleveland and Columbus have been covered, and ventilation is provided by exhausting the air through a tall stack or, in the latter case, with incinerator flue gas. At Ann Arbor (9) and Lansing, Mich. (10), all of the preliminary treatment units are housed in a single building. Pre-chlorination of the sewage for odor control should be helpful for this problem.

AERATION

A marked trend is the tendency toward shorter aeration periods. At the Chicago (North Side) works (3), a battery of aeration tanks was operated without appreciable sacrifice in air economy or quality of effluent for nearly 2 yr with a detention period of between 3 hr and 3.5 hr. The weak character of Chicago's sewage was perhaps reflected in this performance. The design of the New York (Tallmans Island) plant (11) contemplates the use of only two aeration tanks to provide a detention period of 3.5 hr, and provision is made for the addition of a third tank if found necessary. The admission of settled sewage at three points in these tanks, after the introduction of return sludge, may assist in obtaining satisfactory performance. At Columbus, one aeration tank is equipped to operate in this manner for experimental purposes. The application of graduated aeration also tends toward a shorter period than 5 to 6 hr, as usually provided for treating domestic sewage.

Aeration Tanks.—The design of aeration tanks and their details has been described (12) by S. W. Freese, M. Am. Soc. C. E., who discussed many of the related problems and trends. In the arrangement of aeration tanks, a noticeable trend is the piping of one or a group of aeration tanks to a companion unit or group of final settling tanks. Although the idea was expressed in the design of the Charlotte (N. C.) plant (13), it has been expanded. At Charlotte, each aeration tank is constructed integrally with a final settling tank. At Durham, N. C. (North Side) (14), the tanks are similarly arranged except that an equalizing channel permits the use of various combinations when a tank is out of service. In the design of the plants at Baltimore, Md., and Columbus, where the aeration and final settling tanks are separate structures, aerated equalizing channels are provided with stop planks, so that companion units can be sectionalized for comparative operation under different conditions.

To eliminate the possibility of "short circuiting" in the flow channels of aeration tanks, and thereby prevent the leakage of under-aerated solids into the final tanks, occasional cross rows of diffuser plates are installed in several plants with the idea of furnishing agitation of the spiral core. At Baltimore, two vertical baffle walls are placed across each channel to serve the same purpose. The diffuser plates in the aeration tanks at Durham (North Side) are placed lengthwise in the center of each channel with the plates set in a vertical position. This arrangement produces two spirals of flow moving outward from the center at the surface.

Data relating to aeration tanks are given in Table 2. In the smaller activated sludge plants not included in the table, mechanical aeration devices of various types are preferred to diffused air, especially for nominal capacities less than 1,000,000 gal daily (15) (16). Of the works listed in Table 2, only Phoenix, Ariz., and Jackson, Mich. (Items 16 and 17), use paddle agitators combined with diffused air.

Graduated Aeration.—Graduated aeration (more popularly termed "tapered" aeration) appears to be a forward step in rationalizing operating procedure. The principle of "tapered" aeration is founded upon research which will be noted subsequently. It provides that the air applied to the sewage-sludge mixture in an aeration tank shall be graded in accordance with the oxygen-demand curve, with the greatest rate applied near the inlet of a tank. The rate is decreased with the passage of flow through the tank. Probably the first installations to make large-scale use of this principle are at Providence, R. I. (17), and Jackson (18). In the Providence plant, the percentage of diffuser-plate area to tank-floor area is decreased in steps as the aeration period increases for flow through the tanks. In the Jackson plant, the first channel of the aeration tank contains a row of diffuser plates along the wall and the

TABLE 2.—DATA RELATING TO AERATION TANKS IN ACTIVATED SLUDGE PLANTS

Item No. (see Table 1)	Number			Design sewage flow[f] per tank, in million gallons daily	Flow Channels					Diffuser Plates[u]		
	Total	Batteries	Average, per battery		Number per tank[o]	Inside Dimensions, in Feet: Width, each	Length, each	Liquid depth over plates	Total depth	Rows per channel	Number per tank	Ratio of tank area to plate area[cc]
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
1	16	2	8	25.0	4	32.75	434	15.0	17.0	2	2,820	20.2
2	16	4	4	11.25	4	22.25	345	15.0	17.0	2	2,240	13.7
3	36	3	12	4.86	2[p]	16.12[q]	420	15.0	17.0	2	1,440	9.4
4	24	2	12	3.54[g]	2	22.0	236	14.3	16.6	...	2,514	4.1
	12	1	12	5.83	2	21.5	370	14.6	16.6	2	1,316	12.1
5	22	2	11	6.19	1[p]	33.16[r]	425.5	15.0	17.0	1[v]	360 720[v]	39.0 19.5
6	16	4	4	7.69	2	27.0	334	14.5[t]	16.5	2[w]	1,315	13.7
7	10[a]	2	3 7	....	2[i] 4[i,o]	20.0[i] 30.0[n]	238	15.0	16	4[i] 3[n]	1,428[i]	13.3[i]
8	8	2	4	6.25	2	26.0	370	14.4	16.75	2[x]	1,328	14.5
9	4	2	2	10.0	2	30.25	383	15.0	17.33	2	1,232	18.8
10	2[b]	1	2	20.0	4	22.25	373	15.0	...	...	...	...
11	16	4	3 5	2.25	4[p]	15.0	115	10.0[t]	...	...[y]	...	...
12	6	2	3	5.0	3	20.0[s]	150	15.0[t]	16	3	...	14.3
13	4	1	4	5.5	3[p]	22.7	189	15.0[t]	...	...	...	...
14	5	..	..	....[g,h,i]	1[i,p]	18.0	280	9.75	...	...	...	7.0[i]
15	4[c]	1	4	3.0	1	...	...	...	...	...	...	...
16	5	1	5	2.4[j]	1[p]	27.0	330	14.0	...	1	260	34.3
17	4	1	4	2.81[j,h]	2	12.75	240	14.5[t]	16.5	1[z]	...	...
18	33[d]	..	..	0.24[l,g]	1[p]	10.0	67.5	15.0	...	...	...	...
19	4	1	4	2.25	2	15.0	210	15.0	16.5	2	...	...
20	5	1	5	1.5	2	16.0	169	14.0	...	2	...	...
21	4	1	4	1.75	2	15.0	189.5	9.5	11.5	2[aa]	504	11.3
22	4	2	2	1.63	1[p]	17.0	260	14.0	...	...	...	...
23	4	1	4	1.56	1[p]	30.0	135	15.0	17.0	3	336	12.0
24	6	1	6	1.0[m]	1[p]	15.0	200	11.0	...	3	504	6.0
25	6	1	6	1.0[n]	1[p]	...	...	...	...	2[bb]	...	...
26	3	1	3	2.0	2	...	...	...	...	...	...	11.6
27	3	1	3	2.0	2	...	...	...	...	...	...	11.6
28	2	1	2	3.0	3	20.0	139.5	15.0	...	3	...	10.9
29	5	1	5	1.0[m]	1[p]	15.0	200	11.0	...	3	504	6.0
30	4[e]	1	4	1.13	2	15.0	108	14.5[t]	...	2	...	...

[a] Three built in 1936. [b] Provision for third tank. [c] Two primary and two secondary. [d] Annular tanks, arranged about central final settling tank, added in 1938. [e] Each pair arranged to operate in series. [f] Spiral flow in each case except where noted. [g] Ridge and furrow. [h] Spiral flow in one tank. [i] Original tanks. [j] Spiral flow; paddles. [k] In second channel of each tank. [l] Baffles added and alternate air leads shut off to produce cross spiral flow. [m] Longitudinal furrow. [n] Double spiral flow. [o] Arranged in series except as noted otherwise. [p] In parallel. [q] One double width channel, 33.5 ft. [r] One double width tank 67.5 ft. [s] One double width channel, 40.8 ft. [t] Approximate. [u] Containers made of concrete in the case of Items 1, 2, 3, 4, 5, 8, 9, 10, 19, 21; aluminum, Items 6, 7 (new tanks), and 22; cast iron, aluminum lined, Items 24 and 29; and galvanized cast iron, Item 25; in Items 7, 18, 28 the tank floor is grooved. [v] Two rows in six tanks only. [w] Also five cross rows. [x] Also four cross rows. [y] Graduated aeration. [z] In first channel of each tank, 25% in second channel. [aa] Alternate plates in outer row. [bb] Diffusers arranged vertically in center of channel. [cc] Column (5) × Column (6) × Column (7) ÷ Column (11).

second or return channel is equipped with paddle agitators on a central horizontal shaft. This latter channel also contains diffuser plates having an area of only one fourth of that in the first channel.

New equipment for the application of "tapered" aeration consists of a series of carbon tubes attached to an air manifold which can be swung upward out of the liquid in the tank for inspection and cleaning. Diffusers of this type are installed in the partial-aeration plant at Cleveland (Southerly).

Operating Galleries.—Operating galleries extending over the full width of a battery of aeration tanks, located between these tanks and the final settling tanks, are usually provided in the larger plants. In works where return sludge is mixed with settled sewage before entrance into the aeration tanks, mixed-liquor distributing channels from which each tank is fed are integral with the operating gallery. At Columbus and Baltimore, return sludge is introduced at the inlet end of each aeration tank, where it can mix with settled sewage that enters by a separate inlet. With this arrangement the sewage, sludge, and air are metered independently for each tank. The sewage meters are fed from a common channel or conduit integral with the gallery. The gallery also contains the air main, drains, return sludge pipes and all metering equipment.

A departure from the usual practice in providing superstructures for operating galleries has been made at Baltimore where metering instruments and hydraulically operated valve controls are housed in a centrally located return-sludge pumping station. The gallery on either side of this station does not have a superstructure. The metering instruments and valve controls are mounted on control tables of a type somewhat similar to those used in water-filtration plants. There are also instances in which partly housed operating galleries are used.

Flow Regulation.—In the plants previously cited, where sewage, sludge, and air are fed independently to each aeration tank, flow is regulated by the use of hydraulically operated valves inserted in the recovery tubes of the venturi sewage meters and in the individual air pipes. At Baltimore, sludge withdrawn from the final tanks, and sludge returned to the aeration tanks, is regulated by venturi rate-controllers of the type used with water filters, but with provision for flushing. Rate controllers have been used previously for sludge withdrawal at North Toronto, Canada (19). Control of the mixed-liquor flow from each aeration tank in the Cleveland (Easterly) plant (Item 6, Table 2) is regulated by a combination of venturi meter and cone-valve. This usage of regulating and metering equipment (20) is a trend toward the accurate proportioning of flows to the various units as desired.

Return Sludge Pumping.—Although air lifts are used at the Chicago (Southwest) plant (Item 1, Table 2), for economical reasons and because of the large quantity of return sludge to be handled (about 80 mgd), centrifugal pumps are more commonly installed.

To provide for the necessary variations in the rate of returning sludge, either variable-speed pumps or constant-speed pumps, operating against variable head, are necessary. Columbus has the first large plant to use the latter method (Item 8, Table 2). Sludge from the final settling tanks enters a relatively small suction well in which the level fluctuates so as to balance the rate of pumping against the rate of inflow. This fluctuation increases or decreases the total pumping head, permitting the pumps to operate over a range of their combined characteristic curves, thereby varying the total rate of discharge. In order to keep the pumping head and the depth of the suction well to a minimum, pumps having flat characteristic curves are used. This method was adopted with certain modifications for the Baltimore plant (Item 9, Table 2).

Pumping facilities for return sludge in the large plants generally have a maximum capacity equal to 40% or 50% of the average sewage flow. This is exceeded in a few cases.

BLOWER INSTALLATIONS

Air blowers in use are of two general types—centrifugal and rotary positive-displacement. The latter type has been used with direct-connected gas-engine drive, which is favored by the relatively low speed of the blowers. Data relating to blower installations in thirty activated sludge plants are given in Table 3. In the ten largest works, the centrifugal type is used except at New York (Tallmans Island, Item 10, Table 3). Centrifugal blower units are driven either by steam turbines or by motors. Motors are of either the induction or synchronous type, the more expensive, latter type being used for power-factor correction. In some plants of medium capacity where rotary blowers are used, one unit is gas-engine driven and two units are motor-driven, or vice versa.

Flexibility for efficient operation over a wide range of total air capacity is often provided by having units of different capacities. In the case of centrifugal blowers of two sizes, the capacity of each smaller unit is usually from 50% to 75% of that of each larger unit. Since power cost for air blowing is a major part of the total operating cost of an activated sludge plant, increased attention has been directed to the selection of blower capacities. Centrifugal blowers, like centrifugal pumps, operate most efficiently at one point on the pressure-volume characteristic curve which is usually the rating point. The efficiency decreases very little within a limited range of intake volumes greater and less than the rated volume. Therefore, for a proposed installation it is necessary to select the sizes of blowers with proper regard for the probable air requirements—minimum, average, and maximum—to secure an economical arrangement.

Certain new features of centrifugal blower installations consist of: Independent structural supports for the blower units separated from adjacent parts of the blower building by elastic joint material; flexible joints in connecting pipes; cone-type check valves automatically operated; and blow-off valves for either the air discharge manifold or the discharge pipe of each blower. Separation of the blower supports from the building and flexible joints in the connecting pipes are provided as means of eliminating vibration in the building. Blow-off provisions are useful in minimizing the pumping effect when a blower is being added to those already in operation.

Motor-driven centrifugal blowers operate near a synchronous speed of 3,600 rpm and, therefore, require rather elaborate protective electrical interlocks.

TABLE 3.—DATA RELATING TO BLOWER INSTALLATIONS IN ACTIVATED SLUDGE PLANTS

		Installed Capacity, in Cubic Feet per Minute			Larger Units					Smaller Units					
Item No. (see Table 1)	Number of units (1)	Total (2)	Per gallon of sewage (3)	Type of blower‡ (4)	Number (5)	Capacity of each, in cubic feet per minute (6)	Rated discharge pressure, in pounds per square inch (7)	Rated horsepower (8)	Type of driver‖ (9)	Number (10)	Capacity of each, in cubic feet per minute (11)	Rated discharge pressure, in pounds per square inch (12)	Rated horsepower (13)	Type of driver‖ (14)	Ratio, capacity of smaller unit to larger unit (15)
1	3	180,000	0.65	C	3	60,000	7.75	...	S	None	...	...	...	...	...
2	6	227,000	1.82	C	4	42,500	7.75	...	M¶	2	28,500	7.75	...	M¶	0.67
3	7	250,000	2.06	C	4	40,000	7.75	2,160	M**	3	30,000	7.75	1,650	M**	0.75
4	6	220,000	2.04	C	2	50,000	10.0	...	S	4	30,000	10.0	...	S	0.60
5	4	140,000	1.48	C	2	40,000	7.75	1,750	M**	2	30,000	7.75	1,340	M**	0.75
6	5	170,000	2.0	C	3	40,000	7.4	...	M¶	2	25,000	7.4	...	M¶	0.63
7	4	91,000	1.75†	C	1	40,000	...	...	S	3	17,000	8.5	...	S	0.43
8	4	72,000	2.07	C	2	21,000	7.3	900	M¶	2	15,000	7.3	750	M¶	0.72
9	3*	49,500	1.78	C	3	16,500	7.5	...	M¶	None	...	...	...	...	...
10	4	60,000	2.16	R	2	20,000	8.0	800	G	2	10,000	8.0	450	G	0.5
11	3	37,500	1.50	R	3	12,500§	7.0	...	M**·††	None	...	...	...	...	...
12	4	36,000	1.73	C	4	9,000	8.5	440	M**	None	...	...	...	...	...
13	3	35,000	2.29	R, C	1	15,000	8.0	550	G	2	10,000	...	...	M	0.67
14	..	...	...	C	..	...	...	...	...	...	...	...	...	...	...
15	4	6,000	0.72	...	1	2,000	...	...	...	2 1	1,500 1,000	... ...	... ...	... ...	... ...
16	4	4,000	0.48	C	4	1,000	...	...	M	None	...	...	...	...	...
17	3	12,000	1.54	C	2	4,500	7.2	250	M	1	3,000	7.2	150	M	0.67
18	5	20,000	2.62†	R	1	...	...	...	...	...	...	...	...	...	...
19	3	13,000	2.08	R	1	5,000	8.0	240	G	2	4,000	8.0	...	M	0.80
20	3	10,600	2.03	R	1	4,500	...	...	M	2	3,900 2,200	...	...	G	...
21	3	11,250	2.31	R	1	5,000	...	...	M**	2	3,750 2,500	...	...	M	...
22	3	5,250	1.16	R	3	1,750	7.0	...	...	None	...	...	...	...	...
23	2	7,800	1.80	R	..	4,500	8.75	230	G, M	1	3,300	...	...	G, M	0.73
24	3	9,000	2.16	C	3	3,000	8.0	170	M	None	...	...	...	...	...
25	3	4,500	1.08	C	3	1,500	6.0	60	M	None	...	...	...	...	...
26	2	8,800	2.11	R, C	2	4,400	...	...	M**	None	...	...	...	...	...
27	2	8,800	2.11	R, C	2	4,400	...	...	M**	None	...	...	...	...	...
28	2	8,100	1.94	R	2	4,050	8.5	180	G	None	...	...	...	...	...
29	3	9,500	2.74	C	1	3,500	8.0	225	G	2	3,000	8.0	150	M	0.86
30	3	6,500	2.08	R	1	3,000	8.0	120	G	...	2,000 1,500	... ...	100 60	M¶·†† ...	... ...

* Provision for fourth unit. † Approximate. ‡ C = centrifugal blower; and R = rotary blower, positive displacement type. § Low speed capacity, 6,250 cu ft per min. ‖ M = motor type; S = steam turbine; and G = gas engine. ¶ Induction type. ** Synchronous type. †† Two speeds.

In general, the design of large blower installations requires most careful attention if operating economy is to be realized.

FINAL SEDIMENTATION

Experience has shown that the final sedimentation step of the process deserves special attention if a clear effluent, free from floc, is to be secured. With this aim, refinement in the design of final settling tanks has been marked since 1930. A general lowering of settling rates, expressed in gallons per square foot of tank area daily, has resulted. As shown in Table 1, the customary design rate is between approximately 900 and 1,000 gal per sq ft daily on the average mixed-liquor basis. There are a few exceptions in which a rate of 1,200 gal per sq ft daily is used. On the maximum mixed-liquor basis, the settling rate is between 1,600 and 1,800 gal per sq ft daily. Corresponding average detention periods range from 1.7 to 2.6 hr, depending upon the tank depths.

Final Settling Tanks.—Prior to 1930, final settling tanks were constructed square or octagonal in shape. Since that time, either circular or rectangular tanks have been built. The choice has often been the result of alternative bidding. Data relating to final settling tanks in thirty activated sludge plants are given in Table 4. Circular tanks are often arranged in groups of four, fed from a common central well or by individual inlet pipes. On the other hand, rectangular tanks are arranged in batteries with either individual inlet

TABLE 4.—DATA RELATING TO FINAL SETTLING TANKS IN ACTIVATED SLUDGE PLANTS

Item No. (see table 1)	NUMBER			Design sewage flow per tank, in million gallons daily	Shape‖	INFLUENT		INSIDE DIMENSIONS, IN FEET			EFFLUENT CHANNELS IN EACH TANK				Effluent rate per foot of weir, in thousand gallons daily
	Total (1)	Batteries (2)	Average per battery (3)	(4)	(5)	Point of inlet (6)	Direction of flow (7)	Diameter or width (8)	Length (9)	Side liquid depth (10)	Type§§ (11)	Number (12)	Total length of weir, in feet (13)	Tank area per foot of weir, in square feet (14)	(15)
1	32	2	16	12.50	C	Center	Radial	126	...	11.00	P	1	381	33	33
2	32	4	8	5.63	R	Center	Both ends	42.5	179	12.75	C	4	324	24	17
3	42*	3	14	...	S	Side	Cross	77	77	12.75	C	3	443	13	..
					C*	Center	Radial	75	...	12.00	P	1	224	20	..
4	11	2	5, 6	7.73	O	Two sides	Cross	98	98	14.00	C	3	588	15	13
	6	1	6	11.67	R¶	End††	Straight‡‡	84	161.5	13.82	C	7	1,176	12	10
5	15†	2	7, 8	9.06	S	Center	Radial	91	91	12.00	P	1	334	25	27
6	16	4	4	7.69	C	Center	Radial	112	...	12.00	P	1	339	29	23
7	26‡	2	12, 14	...	O	....	....	42	78	...	..	...	...	..	..
					C‡	....	....	60	...	...	..	...	...	..	..
8	8	2	4	6.25	R	End	Straight	60	153	12.50	LC	2 each	672	14	9
9	4	2	2	10.00	C	Center	Radial	126	...	13.33	P	1	381	33	26
10	4	1	4	10.00	R	End	Straight	93.8	134	12.00	L	6	730	17	14
11	5	2	2, 3	7.20	S	Side	Cross	102	102	11.00	..	...	...	..	..
12	4	2	2	7.50	C	Center	Radial	90	...	15.00	P	1	238	27	32
13	4	1	4	5.50	S	Side	Cross	72	72	15.00	..	...	...	..	..
14	4	..	...	...	S	....	....	60	60	...	..	...	...	..	..
15	2	1	2	6.00	C	Center	Radial	75	...	12.00	P	1	...	..	..
16	1	1	1	12.00	S	....	....	140	140	11.50	B	1	350	56	34
17	4	1	4	2.81	C	Center	Radial	70	...	10.00	P	1	212	18	13
18	6§	..	...	1.33	S	Side	Cross	50	50	12.40	..	...	...	..	..
19	2	1	2	4.50	C	Center	Radial	80	...	9.00	P	1	...	..	..
20	4	1	4	1.88	R**	....	Cross	60	60	12.80	P	1	...	..	..
						End	Straight	30	60	12.80	LC	2, 1	...	..	..
21	2	1	2	3.50	S	Side	Cross	70	70	9.87	S	1	...	..	..
22	2	1	2	3.25	C	Center	Radial	75	...	9.00	P	1	236	19	14
23	2	1	2	3.13	C	Center	Radial	70	...	12.00	P	1	220	18	14
24	6	1	6	1.00	R	End	Straight	15	52	10.80	E	1	15	52	67
25	6	1	6	1.00	R	End	Straight	15	50	11.75	C	1	...	..	..
26	2	1	2	3.00	R	End	Straight	44	68	12.00	LC	2, 1	180	17	17
27	2	1	2	3.00	R	End	Straight	44	68	12.00	LC	2, 1	180	17	17
28	2	1	2	3.00	C	Center	Radial	75	...	11.00	P	1	236	19	13
29	5	1	5	1.00	R	End	Straight	15	52	10.80	E	1	15	52	67
30	4	1	4	0.75	R	End	Straight	16	65	11.00	LC	2 each	130	8	6
				1.50	R	End	Straight	33	65	11.00	LC	2 each	200	11	8

* Includes 12 tanks, added 1936–1937. † Space for one more. ‡ Twelve built in 1936. § Additional 78-ft diameter tank in 1938. ‖ C = circular; R = rectangular; S = square; and O = octagonal. ¶ Two revolving suction mechanisms each. ** 1 square, and 3 rectangular. †† Either or both. ‡‡ Or both ends to center. §§ P = peripheral; C = cross; L = longitudinal; B = bracketed launder; S = side; and E = end.

pipes or a common inlet channel. Common outlet channels are usually provided for both types.

Effluent Weirs.—For circular tanks, single peripheral weirs are in general use. Effluent weirs for rectangular tanks have been the subject of considerable variation in design. Four interesting arrangements of these weirs are shown in Fig. 2. Little information to indicate the best arrangement has come before the writer. In the tanks at Columbus (Item 8, Table 4), the weirs are adjustable so that any section, either longitudinal or crosswise, can be raised sufficiently to prevent overflow. Thus, it can be determined which combination of weirs will produce the best results.

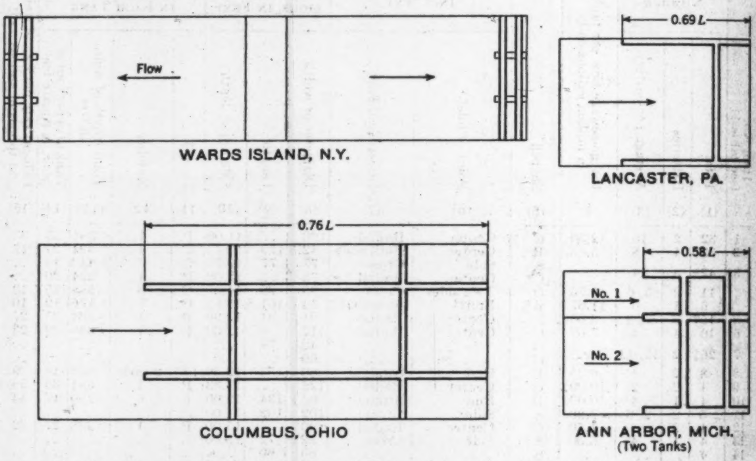

Fig. 2.—Typical Arrangements of Effluent Weirs in Rectangular Final Settling Tanks

A comparison of total weir length, on the basis of tank area per linear foot, gives values from 8 to 24 sq ft in the case of the four plants included with Fig. 2, whereas for the largest circular tanks at Chicago (Southwest) and Baltimore (Items 1 and 9, Table 4) the value is 33 sq ft. However, the Chicago tanks have an overflow rate about 25% greater than those at Baltimore.

Sludge Level Control.—Several devices have been used to determine the depth of sludge blanket in final settling tanks. One of these is the photoelectric cell which was probably first tried at Morristown, N. J. (21). The possibilities have been appreciated and experimental work has been in progress at Chicago where it is expected that sludge-level control can be made largely automatic. At Baltimore and New York (Tallmans Island), the cells are used with indicating lights.

Sludge Disposal

Probably no other branch of activated sludge practice has shown more divergent trends than methods of sludge disposal. The most common method in the smaller plants has been the return of waste sludge to the preliminary

settling tanks, with subsequent digestion of the combined sludge. In the larger plants, various other methods are used, such as barging to sea, heat-drying for fertilizer production, and incineration. Methods of waste sludge disposal in thirty activated sludge plants are noted in Table 5.

TABLE 5.—DATA RELATING TO THE DISPOSAL OF WASTE ACTIVATED SLUDGE

Item	Resettled in preliminary tanks (1)	Concentrated (2)	Digested with raw sludge (3)	Dewatered On sand beds (4)	Dewatered By vacuum filters (5)	Dried or incinerated (6)	Ultimate disposal (7)
1	No	Yes*	No	No	Yes*	Flash-dried incinerated‖ ‖	Ash in fill
2	No	Yes†	No	No	No	No	To sea
3	No	No‡	...	...	...	......	......
4	No	No	No	No	Yes	Heat dried	Commercial fertilizer
5	No	Yes§	No	No	Yes	Flash-dried incinerated	Ash in fill
6	No	Yes‖	Yes‖	No	Yes‖	Incinerated‖	Ash in fill
7	No	No	Yes††	No	No	No	Farmers
8	No	Yes¶	Yes	No	Yes	Incinerated	Ash in fill
9	No	Yes**	Yes	No	Yes	No	Farmers
10	No	Yes**·†	Yes	No	No	No	To sea
11	Optional	No	No	No	Pressed	No	To sea
12	Part	No	Yes	Yes	No	No	Farmers
13	Yes	No	Yes	Yes	No	No	Farmers
14	No	No	No	No	Yes	Heat-dried	Commercial fertilizer
15	Yes	No	Yes	Yes	No	No	......
16	Optional	Yes**	Yes	Yes	No	No	Sold
17	Yes	No	Yes	Yes	No	No	Farmers
18	No	No	No	No	Yes	Dried	Commercial fertilizer
19	Optional	Yes¶	Yes‡‡	No	Yes	Optional	......
20	...	No	Yes	Yes	No	No	Farmers
21	...	No	Yes	Yes	No	No	Farmers
22	...	No	Yes	No	Yes§§	No	......
23	Optional	No	Yes	Yes	No	No	Sold
24	No	No	No	No	Yes	No	Farmers
25	Yes	No	Yes	Yes	No	No	Sold
26	Yes	No	Yes	Yes	No	No	Farmers
27	Yes	No	Yes	Yes	No	No	Farmers
28	Optional	Yes**	Yes	Yes	No	No	.....
29	No	No	Yes	Yes	No	No	Sold¶¶
30	Yes	No	Yes	No	Yes	No	Farmers

* Including some from North side. † Decanted in storage tanks. ‡ Pumped to Southwest and West Side. § In control tanks. ‖ At Southerly. ¶ By centrifuges. ** "Picket-fence" thickening tanks. †† In lagoons or pits. ‡‡ Ground garbage to be digested with sludge. §§ Elutriation. ‖ ‖ With pulverized coal as boiler fuel. ¶¶ To private fertilizer company.

Concentration.—Separate concentration of the sludge prior to digestion is provided for in several large works. At the Cleveland (Southerly) plant (22), sludge from the Easterly plant is concentrated in Dortmund-type tanks. Circular sludge-thickening tanks of a different type, with vertical pickets attached to the arms of the revolving scraper mechanisms, are included in the New York (Tallmans Island) and Baltimore plants (Items 10 and 9, Table 5). These thickeners are used at Topeka (4_b_) and at Phoenix (23) (Items 28 and 16, Table 5). The method was first tried at the Los Angeles (Calif.) Experimental Station (24). It was found desirable to apply chlorine near the surfaces of these tanks to delay septic action, and chlorine installations are provided in the plants mentioned.

Centrifuges of a modified high-speed "cream separator" type are installed at Columbus (five units) and at Lansing (one unit) for the concentration of waste sludge. Each machine has a capacity of about 1,500 gal per hr. In the case of the concentration tanks the decanted supernatant liquor is discharged with the plant effluent whereas in the case of the centrifuges the supernatant liquor is returned to the preliminary tanks.

Digestion.—Digestion with subsequent dewatering of a mixture of raw and waste activated sludge is the most common method of sludge disposal. In plants of medium capacity, it is attractive from the standpoint of effecting operating economies through the use of gas, and in producing a reduced quantity of inoffensive sludge suitable for land use after dewatering on sand beds or by vacuum filters.

Heat-Drying.—The heat-drying of activated sludge in rotary driers to prepare it as commercial fertilizer has been confined principally to the large installation at Milwaukee (Item 4, Table 5) and to the smaller plants at Pasadena, Calif. (Item 18, Table 5), and Houston, Tex. (North Side) (25) (Item 14, Table 5). For a long time, the odor problem from drier gases was acute at Pasadena (26), but subsequently was solved. Prior to heat-drying the sludge is dewatered by vacuum filters.

TABLE 6.—AVERAGE ANNUAL OPERATION

Item No. (see Table 1)	Location	Year	SEWAGE FLOW		PARTS PER MILLION			
			Million gallons daily	Ratio to design flow	Suspended Solids		Biochemical Oxygen Demand[f]	
					Raw sewage	Final effluent	Raw sewage	Final effluent
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
3	Chicago, Ill. (North Side)........	1937	204.8	1.17	140	12	107	9.2
4	Milwaukee, Wis..................	1937	116.3					
	Original works..............		75.4	0.89	278[d]	18	166	8.8
	Extension.....................		40.8	0.58	278[d]	15	166	6.6
5	Chicago, Ill. (Calumet)..........	1937	65.4		151[e]	15	95	13.0
7	Indianapolis, Ind................	1937	49.2	0.76[c]				
	Activated sludge..............		25.3	...	323	14	215	17
	Plain aeration................		23.5	...	323	...	215	89
12	San Antonio, Tex................	1937	16.8	0.56	316	44	190	28
16	Phoenix, Ariz....................	1935	9.94	0.83	177	23.4	170	23.1
17	Jackson, Mich....................	1937[a] 1938[a]	7.01	0.62	178	15	115	2.7
20	Springfield, Ill..................	1937	8.7	1.16	191	12	151	13
28	Topeka, Kan. (East Side)........	1937[b] 1938[b]	5.6	0.93	322	23	302	26

[a] Six months of each year. [b] Eleven months. [c] Approximate. [d] Screened sewage. [e] Iron as Fe, 11 ppm. aeration 8.75 hr. [h] In aeration tanks only. [f] Effluent.

Incineration.—The Chicago (Southwest) (Item 1, Table 5) and Calumet (27) (Item 5, Table 5) plants have notable installations for the incineration of undigested waste sludge after dewatering and "flash"-drying. If the dried sludge can be sold reasonably it may be utilized for fertilizer. On the other hand, the Cleveland (Southerly) and Columbus (28) (Item 8, Table 5) plants include the incineration of digested combined sludge after dewatering. Again, vacuum filters are used for dewatering.

FURTHER TREATMENT

An innovation in the further treatment of the effluent from activated sludge works is weir or cascade aeration for the introduction of additional dissolved oxygen to the effluent. This possibility was observed at Chicago, and a chamber for the purpose is installed at the Southwest plant. An interesting type of cascade aerator, to receive the effluent from a concentric magnetite filter, is provided in the Ley Creek Plant near Syracuse, N. Y. (29). Consideration is being given to the use of magnetite filters for the further reduction (30) of the suspended solid content and the biochemical oxygen demand of activated sludge effluent. These filters have been installed already in several plants where the local conditions have warranted their use. Where seasonal treatment is practiced, by means of double sedimentation without aeration, subsequent filtration of this type offers the means for obtaining an improved effluent.

DATA FOR ACTIVATED SLUDGE PLANTS

Detention period in aeration tanks, in hours	RETURN SLUDGE (PERCENTAGES)		Percentage of suspended solids in mixed liquor	Air, in cubic feet per gallon of sewage	Settling rate of mixed liquor, in gallons per square foot daily	FINAL EFFLUENT (PARTS PER MILLION)		OVER-ALL REMOVAL (PERCENTAGES)	
	Sewage flow (10)	Solids (11)				Dissolved oxygen (15)	Nitrates (16)	Suspended solids (17)	Biochemical oxygen demand[f] (18)
4.2	22.5	1.04	0.23	0.35	1,290	2.0[m]	2.66	91.5	91.5
6.0	32.3	1.61	0.40	1.26[h]	978	...	4.5	93.5	94.8
9.74	29.3	1.76	0.41	1.21[h]	666	...	4.5	94.5	96.0
5.3	22.5	1.48	0.269	0.38	1,063	7.3[n]	4.43	90.1	86.3
10.46	27	1.04	0.26	1.25	617[l]	...	...	95.6	92.1
9.94	None	...	...	0.41	894[l]	...	...	...	58.5
7.56	44.4[i]	...	...	1.00	689[l]	1.7	1.2	86.0	85.5
5.7	29.3[j]	0.88	0.167	0.30	632[l]	2.3	2.13	86.6	86.4
[e]...	15	2.2	0.332	0.8	...	6.7	10.1	92	98
6.4	23	0.87	0.189	0.58	1,200	...		93.4	91.4
5.3 to 6.9[g,h]	27.3 to 39.4[h]	0.21 to 0.44[h]	0.08 to 0.156[h]	1.14 to 1.53[h]	...	1.0 to 2.9[h]	...	93	91

[f] Five-day at 20° C. [g] Re-aeration 3.9 to 4.9 hr. [h] Monthly averages. [i] Re-aeration 1.12 hr. [j] Re-
[m] At outfall sewer, 8.9 ppm. [n] At outfall sewer.

OPERATION AND CONTROL OF PROCESS

The operation of activated sludge works and the control of the process have presented a variety of problems. Over a period of more than fifteen years, experience and knowledge have gradually accumulated from the operation of large works. As the result, an approach has been made to the standardization of operating procedure and technique. However, much work remains to be done in this regard.

In general, the results obtained from the use of this process have been satisfactory and have suited the requirements of various localities (7) (9) (18) (31) to (36). Average annual operation data for nine representative activated sludge works are given in Table 6. In these plants, activated sludge operation is continuous through the year. Certain other works are operated on a seasonal basis with aeration omitted during months when relatively high flows prevail in the receiving streams. J. R. Collier, of Elyria, Ohio, and E. E. Smith, Assoc. M. Am. Soc. C E., have both reported (37) that their respective plants were operated with seasonal aeration, and that by careful control the annual starting of the process presented no abnormal difficulties. Of course, operating economies in power consumption are achieved by seasonal treatment.

Starting Procedure.—During the period of initial operation and adjustment of an activated sludge plant, many unexpected problems arise which test the skill of the operating force. Valuable lessons are learned which contribute to the advancement of design. Interesting experiences of starting operation at Ann Arbor (9), Lancaster, Pa. (two plants) (38), Peoria, Ill. (39), and elsewhere, have been reported.

The concensus of opinion of Ohio operators (37) regarding the preferred method for producing activated sludge (undoubtedly for small plants) is briefly, as follows: (1) To fill the aeration tank with freshly settled sewage and to aerate for a period; (2) to settle this aerated sewage and decant the clear portion to the final settling tank; (3) to replace the decanted liquor with settled sewage, and to repeat the operation until enough activated sludge has been produced to satisfy the requirements of incoming sewage.

About one week is usually required to establish the process and to develop satisfactory activated sludge. At the Lancaster (North) plant, three weeks were required during cold weather.

Control of Process.—Control of the process has shown a distinct trend toward simplification. Valuable criteria of performance are the suspended solids in the mixed liquor, the dissolved oxygen in the liquid of both the aeration and final settling tanks, and the sludge index and microscopic examination as warnings of the possible "bulking" of the sludge. In the application of these criteria, the tendency has been to reduce the time element so that corrective steps can be taken when needed. The technique has been simplified (40) (41) so that any intelligent shift operator can perform the necessary tests.

Suspended Solids in Mixed Liquor.—The optimum suspended-solid content of the mixed liquor in the aeration tanks is best determined by actual plant performance. It is related to the character of the sewage to be treated, the quantity of air introduced for the maintenance of biological activity, the detention period, and possibly other factors. The rate of returned sludge required to maintain a suspended solid content will vary, of course, with the percentage of solids in the returned sludge. This relationship is shown by the curves in Fig. 3, with the average suspended solid content of the mixed liquor superimposed for each of seven plants (the curves neglect the suspended solids in the settled sewage). The wide variation in the practice of maintaining solids in the aerated mixture is evident and apparently stresses the influence of local operating conditions. In this regard, the 1937 report on sewage disposal at Indianapolis (36) states that mixed-liquor solids were held at approximately 0.25% and that over-aeration, as shown by turbid effluent and foaming in the aeration tanks, resulted when the solids reached 0.30%.

In a letter dated December 23, 1938, Wellington Donaldson, M. Am. Soc. C. E., states that the suspended solids in the New York (Wards Island) aeration tanks were reduced from 1,500 ppm to 1,300 ppm (0.13%) in the belief that the lower value would permit both air economy and a larger margin for sludge storage when necessary.

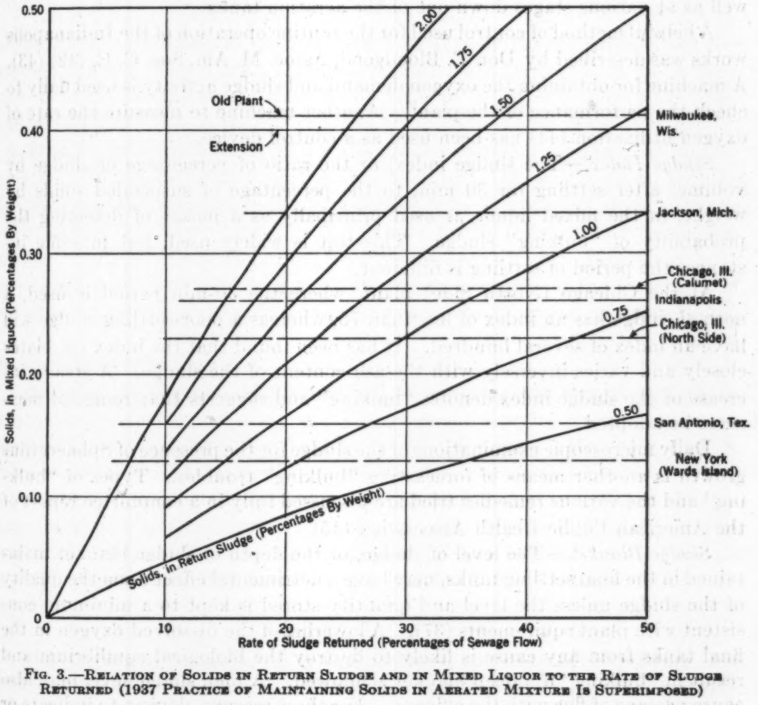

FIG. 3.—RELATION OF SOLIDS IN RETURN SLUDGE AND IN MIXED LIQUOR TO THE RATE OF SLUDGE RETURNED (1937 PRACTICE OF MAINTAINING SOLIDS IN AERATED MIXTURE IS SUPERIMPOSED)

The laboratory centrifuge is used in numerous plants to determine the percentage of suspended solids rapidly. The values obtained from the calibrated centrifugal readings are later checked against gravimetric determinations.

Dissolved Oxygen.—The maintenance of dissolved oxygen in the effluents of both the aeration and final settling tanks, to the extent of at least 2 ppm, is recognized as a necessity for proper operation. In some plants a higher value has been found desirable. In notes dated December 22, 1938, L. M. Johnson, engineer of maintenance and operation, the Sanitary District of

Chicago, with reference to the dissolved oxygen test, states that it is their practice to collect samples in wide-mouthed quart bottles to which about 2 cu cm of a 10% copper-sulfate solution has previously been added. Immediately, the copper sulfate retards the oxygen demand of the sludge in the sample, and also coagulates it for rapid settling. As soon as the sludge has settled sufficiently, the supernatant liquor is siphoned into an 8-oz bottle and treated with the Winkler reagents. Instead of titrating, these bottles are compared colorimetrically with known standards. Mr. Johnson's experience has indicated that dissolved oxygen determinations should be made every 4 hr. Samples are collected at the discharge end of one of the aeration tanks in each battery, as well as at various stages down one of the aeration tanks.

A helpful method of control used for the routine operation of the Indianapolis works was described by Don E. Bloodgood, Assoc. M. Am. Soc. C. E. (42) (43). A machine for obtaining the oxygen demand and sludge activity is used daily to check the performance of the plant. Another machine to measure the rate of oxygen utilization (44) has been used as a control device.

Sludge Index.—The sludge index, or the ratio of percentage of sludge by volume, after settling for 30 min, to the percentage of suspended solids by weight, in the mixed liquor, is used principally as a means of detecting the probability of "bulking" sludge. This test is widely used, but in some instances the period of settling is different.

At the Chicago (North Side) plant, where the 30-min period is used, a normal sludge has an index of less than 75, whereas a poor-settling sludge will have an index of several hundred. It has been found that the index correlates closely and varies inversely with the ash content of the sludge. A steady increase of the sludge index denotes "bulking" and suggests that remedial measures be adopted.

Daily microscopic examinations of the sludge for the presence of Sphaerotilus growth is another means of forecasting "bulking" troubles. Types of "bulking" and the various remedies tried are discussed fully in a committee report of the American Public Health Association (45).

Sludge Blanket.—The level of sludge, or the depth of sludge blanket maintained in the final settling tanks, may have a detrimental effect upon the quality of the sludge unless the level and quantity stored is kept to a minimum consistent with plant rquirements (37). A lowering of the dissolved oxygen in the final tanks from any cause is likely to destroy the biological equilibrium and result in "bulking" if a deep blanket is allowed. A high sludge level may also cause passage of floc with the effluent. For these reasons, devices to indicate or to control the sludge level should be useful.

The density of sludge, as previously stated, is related to the rate of returned sludge required to maintain a pre-determined, suspended-solid content in the mixed liquor (Fig. 3). It is probable that increased depth of sludge blanket, with consequent compaction, produces a greater density of the sludge. Therefore, relatively deep draw-off hoppers of limited capacity and continuous stirring may be advantageous. To attain this end, the circular final settling tanks at Baltimore have a steeper bottom slope near the center than near the periphery.

The stirring mechanism, within the limits of the center cone, is equipped with "pickets" for thickening.

Disturbing Influences.—It is well known that certain influences have a harmful effect upon the satisfactory performance of activated sludge works. Some of these factors are septic raw sewage, variations in the quantity and strength of the incoming sewage, the return of strong supernatant liquid from digestion tanks to the sewage ahead of preliminary sedimentation, and relatively large proportions of certain industrial wastes in the sewage. The effect of septic sewage may be reduced by pre-chlorination in the outfall sewer upstream from the plant.

The production of a higher quality supernatant liquid by the use of two-stage digestion should aid to some extent in overcoming difficulties from this source. In the New York (Tallmans Island) plant, provision is made for a 6-hr detention of the supernatant liquid prior to its introduction to the plant influent (11).

With regard to industrial wastes the process has been used successfully for treating sewage containing appreciable quantities of wastes from packing houses, vegetable canneries (43), rayon mills (46), and other sources. On the other hand, wastes from metal works, dairies, textile mills, and diverse industries have proved troublesome, and in some cases such known wastes may have precluded the use of the process. Noteworthy research on the treatment of sulfur dye wastes with sewage by the activated sludge process was undertaken by the Textile Foundation on a semi-plant scale (47) and demonstrated the successful use of the process.

Economies.—The practice, since 1930, of reducing the quantity of air to effect operating economies and thereby decrease the nitrate content of activated sludge effluents was discussed (48) by F. W. Mohlman. He noted that in cases where additional downstream dilution does not occur for a long period, nitrification of ammonia contained in the plant effluent may result in the disappearance of dissolved oxygen in the stream to the detriment of fish life. The retardation of luxuriant growths of algae is perhaps another benefit derived from the maintenance of a lower content of nitrate in the effluent.

Fundamental Research

The perplexing character of many problems encountered in practice, together with the desire for a more complete understanding of the fundamental phenomena and the environmental factors of the activated sludge process, has stimulated research in this field.

Important findings of the numerous studies undertaken by the Division of Stream Pollution Investigations, U. S. Public Health Service ((49) to (52), inclusive), the Sanitary District of Chicago (53), the Division of Water and Sewage Research, New Jersey Agricultural Experiment Station, and other research groups (44) (54) (55) were reported from 1930 to 1938. Although the complete theory of the process has not been established, the studies just mentioned have contributed greatly to this end.

NEEDED STUDY AND RESEARCH

Because of the comprehensive scope of the subject and the wealth of related data, this paper has been necessarily confined to a general discussion of the problems and trends in activated sludge practice. Detailed consideration of specific problems has been omitted. The need for additional study and research is evident. Certain phases for further investigation, leading toward the standardization of practice, are noted as follows:

(1) The influence of basic factors such as the character of sewage, variation in flow, and the degree of pre-treatment upon plant performance, the object being to determine design limits.

(2) The effects of certain industrial wastes and sewage upon normal functioning of the activated sludge process, and proper means for the adjustment of the process or for partial treatment of the wastes at their source.

(3) Desirable methods for the treatment of the supernatant liquid from digestion tanks prior to introduction of the liquid to activated sludge treatment devices. Determine the best point of application.

(4) Comparison of several methods for thickening activated sludge prior to digestion, and the formulation of desirable operating procedure.

(5) Starting procedure for the process with recommendations as to suitable practice particularly for small plants.

(6) Parallel comparison of graduated aeration and uniform aeration, to determine relative advantages, under similar conditions of routine operation.

(7) The possible general relation between the optimum suspended-solid content of aerated mixtures and the character of settled sewage.

(8) The annual increase in air-pressure loss for diffuser devices. The recovery of pressure loss following the cleaning of these devices, the results preferably expressed as percentages of the original loss. Tests to be performed under normal conditions of continuous operation.

(9) From the preceding study, to establish both the economical periods of cleaning and useful service for diffuser devices of common porosities, consideration being given to the balancing of costs for cleaning, replacement, and power consumption. These data should be helpful in selecting the economical rating pressure for centrifugal blower installations.

(10) Extension of early studies on measured air losses for pipes of various sizes and different materials to obtain suitable design coefficients for use with the Fritzsche formula. Measurement of the air-pressure loss in fittings and valves of various types.

(11) The minimum quantity of free air required for the satisfactory agitation of aerated channels having various depths, with results expressed in terms of cubic feet per minute per square foot of channel surface.

(12) The efficiency of final settling tanks and factors affecting their design. Consideration of inlets and overflow weir arrangements.

(13) The relation of depth of sludge blanket to the density of activated sludge in final settling tanks.

(14) The rate of absorption of oxygen by effluents when subjected to cascade or weir aeration with various heights of fall, degrees of agitation, or surface area exposed.

(15) Improvements to the methods and technique of process control through a continuation of research relating to the fundamental phenomena and theories involved in the process.

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(53) **"The Oxygen Requirements of the Activated Sludge Process,"** by S. Grant, E. Hurwitz, and F. W. Mohlman, *Sewage Works Journal*, Vol. 2, No. 2, April, 1930, p. 228.

(54) **"The Biology of Activated Sludge. An Historical Review,"** by A. M. Buswell, *Sewage Works Journal*, Vol. 3, No. 3, July, 1931, p. 362.

(55) **"Research in Sewage Chemistry, Sewage Treatment and Stream Pollution. A Critical Review of the Literature of 1937,"** by Charles A. Emerson, Jr., Gail P. Edwards, H. F. Gray, Willem Rudolfs, and Harold W. Streeter, Members, Am. Soc. C. E., and Harry A. Faber, H. Heukelekian, and Earle B. Phelps, *Chairman*, *Sewage Works Journal*, Vol. 10, No. 2, March, 1938, p. 173.

(56) **"The Tri-Cities' Activated-Sludge Plant at Alhambra, Cal."** (anonymous), *Engineering News-Record*, Vol. 95, No. 18, October 29, 1925, p. 714.

(57) **"Distinctive Characteristics of the Indianapolis Sewage Treatment Plant,"** by Charles H. Hurd, M. Am. Soc. C. E., *Sewage Works Journal*, Vol. 1, No. 5, October, 1929, p. 578.

(58) **"Large Activated Sludge Plant for San Antonio, Tex.,"** by H. R. F. Hellend, M. Am. Soc. C. E., *Engineering News-Record*, Vol. 105, No. 23, December 4, 1930, p. 886.

(59) **"Wards Island Sewage Treatment Plant Begun,"** by George W. Fuller, *Civil Engineering*, Vol. 1, No. 14, November, 1931, p. 1260.

(60) **"The New-Activated Sludge Plant at Phoenix,"** by Dario Travaini, *Sewage Works Journal*, Vol. 4, No. 3, May, 1932, p. 525.

(61) **"The San Antonio Sewage Treatment Plant,"** by W. S. Stanley, *Water Works and Sewerage*, Vol. 80, No. 1, January, 1933, p. 21.

(62) **"Sewage Collection and Disposal at Lancaster, Pennsylvania,"** by John F. Laboon, *Sewage Works Journal*, Vol. 5, No. 6, November, 1933, p. 1021.

(63) **"The Lima Sewage Treatment Plant,"** by W. S. O'Brien, *Sewage Works Journal*, Vol. 6, No. 1, January, 1934, p. 42.

(64) **"Milwaukee Extends Facilities for Sewage Treatment,"** by Darwin W. Townsend, M. Am. Soc. C. E., *Engineering News-Record*, Vol. 116, No. 5, January 30, 1936, p. 153.

(65) **"The Wards Island Sewage Treatment Project,"** by Walter D. Binger, M. Am. Soc. C. E., and Richard H. Gould, *Municipal Sanitation*, Vol. 8, No. 12, December, 1937, p. 627.

DISCUSSION

Langdon Pearse,[3] M. Am. Soc. C. E. (by letter).—A useful compilation of statistical data on existing activated sludge works is presented in this paper, but it falls short of picturing the art accurately and of giving a weighted appraisal of current values in its practice. Furthermore, the author makes suggestions, without comment, for trial. With this preamble, a few comments may be helpful.

North Side Works.—The North Side Works of the Sanitary District of Chicago is cited as an example of treating a sewage flow considerably in excess of its design capacity. The plant, however, was planned hydraulically to care for a considerable excess flow over the capacity assigned in 1928, so that by the addition in 1937, of a few final settling basins, a plant rated in 1928, on average capacity, at 180 mgd is now rated at 250 mgd. As this was the first plant ever constructed to handle an average flow of more than 180 mgd, the various parts were liberally designed to permit, later, the demonstration of possible economies which were hoped for, although not then established by current practice. Thus a nominal 6-hr aeration period was reduced to 5 hr, and for more than one year one battery (of three) operated at a 3-hr period (see Table 7).

TABLE 7.—North Side Sewage Treatment Works Aeration Period and Air Consumption,* 1937

Month	Aeration Period, in Hours			Quantity of Air, in Cubic Feet per Gallon of Sewage			Final Effluent 5-Day B.O.D., in Parts per Million		
	A	B	C	A	B	C	A	B	C
January	4.7	2.8	4.9	0.28	0.28	0.31	10.9	12.1	11.8
February	5.0	3.1	5.1	0.38	0.35	0.35	10.8	12.0	9.9
March	5.4	2.9	4.5	0.43	0.36	0.38	9.8	11.3	9.6
April	4.5	2.5	4.1	0.31	0.29	0.30	7.1	8.7	7.4
May	4.2	2.6	4.0	0.34	0.33	0.35	8.0	8.7	9.0
June	4.6	2.9	4.7	0.39	0.38	0.38	5.8	8.4	6.9
July	5.0	3.2	4.5	0.38	0.40	0.36	6.5	7.1	7.9
August	5.0	2.9	4.3	0.39	0.39	0.36	7.4	6.6	7.1
September	4.4	2.9	4.1	0.37	0.38	0.34	9.5	8.4	11.2
October	4.5	2.9	4.5	0.31	0.35	0.34	7.2	7.2	7.4
November	5.3	3.1	4.5	0.38	0.33	0.33	10.8	10.2	9.7
December	6.0	4.2	5.3	0.40	0.34	0.35	14.4	13.6	12.4
Average	4.9	3.0	4.5	0.36	0.35	0.35	9.0	9.5	9.2

* A, B, and C refer to Batteries A, B, and C.

Grit Chamber.—With the adoption, by the Sanitary District, of dewatering and incineration of sludge, the grit chambers were omitted at the Calumet and Southwest works because it was thought that the grit could be handled in the preliminary tanks. At the Calumet works, the preliminary tanks were designed on a 10-min period, but actually have been working at periods of about 20 min. However, the plants, as constructed, include a hydraulic gap for the grit chambers. Operation at the Calumet works has demonstrated the need of a grit chamber under Chicago conditions, particularly for use during storms. In 1939, grit chambers were installed at the Calumet works on the sludge removed from the preliminary tanks—1 mgd for each of two units. They are cleaned mechanically, with flexibility in feed and in varying effluent weir length.

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[3] San. Engr., The San. Dist. of Chicago, Chicago, Ill.

Sludge Handling.—Whether the method of sludge handling by the Sanitary District is correctly described as a "use of waste heat" is a question, inasmuch as there is practically no waste heat in the dewatering and incineration of sludge in the District under normal conditions. The heat recovered by burning the sludge (when burned) is used in drying the wet cake. The generation of steam depends primarily on the use of additional coal. To avoid odors, the gases to be vented to the atmosphere are passed through a heat zone of 1,500° F. Thus it was logical design for a large project like the Southwest Works to run the blowers, pumps, and electric generators by steam turbines, and make the boiler fire-box a means of deodorizing. Heat-dried sludge to be sold as fertilizer requires about one half ton of coal to dry one ton of sludge. The economy in operation is not primarily derived from heat recovered as a by-product, but from consolidating the heat-drying and boiler operations under a common operating crew.

Preliminary Treatment.—The use of fine screens should have been noted for preliminary treatment, particularly for a situation like that at Milwaukee, where all visible solids of sewage origin were kept out of the lake. At Chicago, preliminary settling was installed at the North Side, originally, to reduce the volume of sludge pumped through a 17-mile pipe line and, by removing a part of the coarser solids, to reduce the quantity of air required. Actual tests on a plant scale, however, indicate that somewhat less air is required. Preliminary treatment is necessary on such a weak sewage, although probably not in proportion to the solids removed. Also, preliminary settling was used at the Calumet and the Southwest works because large-scale tests demonstrated that a mixture of fresh and activated sludge dewatered more readily on a vacuum filter than activated sludge alone.

Aeration.—Under "aeration," there are a number of points worth consideration. As to aeration period, on a weak sewage at Chicago (West Side, settled), in experimental work, the aeration period was reduced to less than 2 hr before the quality of the effluent began to deteriorate. In a number of installations in the United States, many devices or arrangements have been tried, such as introducing settled sewage at three points in an aeration tank, after the introduction of return sludge, and graduated or tapered aeration. As yet the value of such procedure has not been demonstrated.

Aeration Tank Design.—Cross baffles have been used in Great Britain to a limited extent, and were installed at Pasadena, Calif., many years ago. Later they were used at Tenafly, N. J., and Hagerstown, Md. In the design of the Sanitary District plants, long runs have been provided in the aeration tanks.

Diffuser Plates.—An important trend, omitted by Mr. Regester, should be noted in the rating of diffuser plates. Early installations used porous plates with a porosity around 9 to 18 cu ft of air per min per sq ft at 2-in. water pressure (Milwaukee, 9 to 13; North Side, Chicago, 14 to 18). Since 1929, the tendency has been toward more porous plates at Indianapolis, Milwaukee, Chicago, and elsewhere. Those purchased for the Southwest (in 1936) and Calumet works (in 1934) ranged from 36 to 44 cu ft per min. These higher porosities are favored on the theory of lower friction losses and reduced clogging. However, more than 90% of the original plates installed in the North Side Works in 1928 were still in service 11 years later, with comparatively slight increments in pressure loss. However, at Chicago, sudden clogging has occurred in experimental work on tannery wastes due to the deposit of calcium carbonate on the plate surface, and in operating the new works at Calumet, clogging was due to ferric oxide deposit on the surface of the plates, formed from spent pickling liquors high in ferrous iron which entered the sewage.

Flow Regulation.—At the North Side and Calumet works, the sludge flow from individual settling tanks is controlled manually, by the setting of a gate valve at the former plant and by the overflow of a fountainhead regulator at the latter. No venturi rate controllers are used. At both plants, the sludge from the final settling tanks is discharged into a common channel in each battery, and thence to the return-sludge pumping station, where a return-sludge pump for each battery, plus one spare pump, is provided. The pumps are driven by variable-speed motors. Originally the speed of these motors was controlled by electrolytic controllers, actuated by a float in the pump suction well. However, at the North Side, these have been replaced by manually-operated resistance-type controllers. At each plant the return-sludge flow from each battery is metered and then passed to the sewage conduit leading to the aeration tanks. The mixed-liquor flow from each aeration tank is metered at both plants.

At the Southwest works, the sludge is removed from each final settling tank by a pair of air lifts, the sludge from all tanks in each battery discharging into a common channel leading to the mixing channel. The return-sludge flow from each battery is metered. The quantity of sludge from individual tanks is estimated from the air used by the air lifts. The air to the aeration tanks is regulated by means of globe or angle valves at all three plants. The flow to each half tank at the Southwest works, and the total flow to each battery of tanks, are metered.

The use of rate controllers on mixed-liquor flow was canvassed years ago at Chicago, but was discarded as being somewhat too refined.

Operating Gallery.—The utilization of an operating gallery in activated sludge practice was developed by the writer in 1920, from his earlier experience in designing rapid water filter plants. In large plants, particularly in northern climates, there is a need for operating galleries. The housing has been reduced in size at Calumet and Southwest. At the North Side, a number of meters and recording devices that were installed originally have been abandoned, after extended trial.

Air Cleaning.—The cleaning of air before compression is important. Water washing has been found to be of little value. Oil-coated media used as a filter are commonly favored, whether in fixed frames or moving continuously, being dipped in an oil bath.

Blower Installation.—The design of a blower installation depends on the size of plant and type of blower. Where centrifugal blowers are used, the use of a blow-off is almost unknown. With Connersville or "tub" types, a blow-off may be highly desirable.

Final Sedimentation.—Final sedimentation tanks have increased in size since the San Marcos plant was put in operation in 1915. The need of weir study on activated-sludge final settling tanks was first stressed at Chicago and Springfield, Ill. Actual tests were made by S. I. Zack, M. Am. Soc. C. E., under the writer's direction about 1931 at the Des Plaines River Treatment Works of the Sanitary District. The H-weirs and end weirs set back from the end wall were developed at that time. Supplemental weirs or gutters had been installed as early as 1919 at Houston, Tex., by G. L. Fugate. Tests on the original North Side design have shown that the final settling tanks are satisfactory, with flow admitted to a square tank from two opposite sides, the effluent being taken over the weirs of three cross troughs. Before the Calumet plant was designed, S. L. Tolman made an extended series of tests at Chicago, Milwaukee, and San Antonio, Tex., for the Sanitary District, to determine allowable rates of flow.

Various devices to determine levels of the sludge blanket have been made. So far as the writer knows, the first use of the photo-electric cell was at the Des Plaines River Works in 1926. This device was designed to give an alarm if the sludge level rose too high. Later, attempts were made to connect the cell to control devices. From a practical standpoint, the use of small spouting air lifts reaching to different depths has proved satisfactory in all the plants of the Sanitary District.

Sludge Disposal.—In the question of sludge disposal, W. L. Stevenson, M. Am. Soc. C. E., first indicated the possibility of digesting fresh and activated sludge in an Imhoff tank.[4] So far as the writer knows, the first use of this procedure was in the Calumet plant of the Sanitary District, in 1922. For the small plants, the digestion of mixed sludge offers a suitable method of handling the sludge. For the large plants, like the Calumet, Southwest, and North Side of the Sanitary District, other procedures were also studied, resulting in the development of the dewatering and flash-drying system, with incineration or disposal of the heat-dried sludge for fertilizer. Since the Southwest works began production of heat-dried activated sludge in the autumn of 1939, orders for more than 39,000 tons had been booked for future delivery, prior to June 30, 1940. The Milwaukee production was "sold out" for the same period. The first 6,700 tons delivered from the Southwest works averaged about 6.46% ammonia, and grossed $8.07 per ton, f.o.b. Chicago.

Conditioning of Sludge.—The conditioning of activated sludge for vacuum filtration was made practical by the discovery, at the old Calumet works, of the value of ferric chloride. Through the cooperation of the chemical industry, the price of ferric chloride has been reduced steadily. Bids on considerable tonnage received in 1939 show a low of 1.21 cents per lb of anhydrous ferric chloride, or with freight on water included, approximately 1.8 cents, delivered. This is a remarkable reduction from about 8 cents per lb current prior to 1925.

Centrifuges.—The application of centrifuges to dewatering sewage sludge has proved a "headache" because of the difficulty of obtaining adequate removals of solids at a reasonable cost. The Sanitary District made tests about 1932 on a sufficient scale to demonstrate the requirements to produce the desired results in removal of solids. Working plans were made for practical units, but a machine was never built because the procedure was too expensive. However, centrifuges of batch laundry type are used to dewater screenings at Milwaukee.

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[4] *Proceedings*, Am. Soc. of Municipal Improvements, 1916, pp. 460–463.

Final Filtration.—For further treatment, the value of filtration following final sedimentation has yet to be demonstrated as a general procedure.

Seasonal Operation.—The Indianapolis plant for many years has been operating on a basis of a seasonal operation, whereby in the colder months the major part of the plant uses aeration without sludge return, only a minor portion having sludge return. From June to October as much sewage as practicable is given complete treatment, the amount varying with the conditions each year.

Starting of New Plants.—Probably the first of the largest plants in the United States was the Houston South Works, started in 1917, following the discovery by Edward Bartow, M. Am. Soc. C. E., and F. W. Mohlman, at the University of Illinois[5] in 1915, that an activated sludge plant could be started off on a continuous basis, and sludge developed in 10 to 15 days showed marked purification. This eliminated the use of a fill and draw or batch process. So far as the writer can ascertain, all the later works of any material size started off along the lines laid down by Messrs. Bartow and Mohlman. Possibly the use in Ohio, by small plant operators, of the fill and draw process for starting may be justified, but the reasons for such procedure should be made clear.

It is hoped that Mr. Regester will report what Mr. Donaldson accomplished by a proposed procedure to reduce the mixed liquor solids from 1,500 ppm to 1,300 ppm. It seems questionable whether such a change would make an appreciable difference in operating results.

Effect of Depth on Concentration of Sludge.—In the discussion of the effect of depth on concentration of activated sludge, Mr. Regester states: "It is probable that increased depth of sludge blanket, with consequent compaction, produces a greater density of the sludge." A few tests, made in 1931 by the Sanitary District, on different depths of tanks indicate roughly that the density of activated sludge may vary but little with depths from 10 to 30 ft. For that reason, various devices were explored to aid concentration. A picket fence concentrator (Dorr type) was tried at the Des Plaines River Plant in the early fall of 1931.

Industrial Wastes.—The reference to the application of activated sludge to industrial wastes is necessarily sketchy. The Sanitary District has operated testing stations on packing house, corn products, and tannery wastes. Others have given much study to different industries. Out of this study (which began as early as 1916) has grown a feeling that treatment of industrial wastes mixed with domestic sewage in a municipal plant may prove a practical solution of the problem in many situations. In the case of the corn products industry, bottling up the waste waters and concentration of solids, with removal by vacuum filtration, together with other operating adjustments in the manufacturing procedure, proved a very practical solution.

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[5] *Bulletin No. 14*, Univ. of Illinois, Urbana, Ill., pp. 325–335.

Sampling.—One important feature of plant practice, notably for activated sludge, but also all sewage works operation, is adequate and reliable sampling. From an experience in the work on corn products waste in 1920, the Sanitary District has developed a practice of using automatic samplers as far as practicable. The earlier samplers were home-made. Later types were developed by ingenious manufacturers.

Desiderata.—The list of fifteen desiderata, which Mr. Regester catalogs, includes many on which a considerable fund of information is available. Much of this information has been collected by operating workers from actual practice, and has been described in part in published articles and annual reports. Some of the questions he raises involve not only activated sludge, but the entire field of sewage treatment and industrial waste handling. From more than 30 years' experience with The Sanitary District of Chicago, the writer is firmly imbued with the idea of following up works operation carefully, making large-scale tests as far as practicable, and determining the relative value of various procedures, as to end results, costs, and efficiency. If the designer will seek the criticism and comment of the operating staff, better designs and more economical operating conditions should result.

FRANK C. ROE,[6] ESQ. (by letter).—The special topic of porous diffusers as used in activated sludge treatment is not included in Mr. Regester's very excellent digest of the subject. It is believed that the following may therefore be a helpful supplement.

The first diffusers were perforated pipe. Obviously, large air bubbles resulted in low efficiency and there were corrosion troubles. After trying porous wood diffusers in experimental work at Milwaukee, ceramically bonded silica in the form of porous plates was used successfully. Until 1920, relatively fine grades of such porous plates were used on the theory that the small air bubbles diffused permitted maximum absorption with high efficiency.

In the 1920's, clogging troubles with fine-grade porous plates, resulting in high pressure losses (which became prohibitive in some cases), led to the adoption of porous diffusers having a more open structure. By 1930, the efficacy of coarser porous diffusers was apparent, and specifications have since called for permeability ratings of from 30 to 50 as compared with 10 to 15 prior to 1920.

The dry permeability rating of diffusers (cubic feet per minute of free air per square foot under a 2-in. water pressure) which was arbitrarily adopted in the early days for convenience of testing, has undergone refinement with the introduction of coarser, low pressure-loss plates. Obviously, the control of uniform diffusion becomes more delicate as pressure loss is minimized. Individual diffuser units controlled by a single valve must therefore have the same pressure loss within narrow limits to assure uniformity.

Diffuser manufacturers found that the temperature of plates and testing air must be controlled to avoid errors. Diffuser specifications should be so written that such control is exercised.

Diffuser tubes have proved equally effective in comparison with diffuser plates. In some instances, the assembly and installation of diffuser equipment favors tubes over plates on a cost basis. In general, tubes have proved most beneficial in small plants because of the ease of servicing diffusers. However, tubes have been adopted for a large plant at Gary, Ind., on the basis of highest over-all economy.

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[6] Sales Mgr., Porous Products and Laboratory Ware, The Carborundum Co., Niagara Falls, N. Y.

Under the heading, "Needed Study and Research," Mr. Regester recommends studies on maintaining low normal pressure losses through diffusers. Efforts have been made in several instances along this line. Regular addition of chlorine to air proved effective at Charlotte, N. C. One of the most recent promising discoveries is the process of sandblasting the surface of diffusers where clogging is severe.

GERARD A. ROHLICH,[7] JUN. AM. SOC. C. E., AND CLAIR N. SAWYER,[8] ESQ. (by letter).—The compilation of data relating to the design and operation of activated sludge plants in the United States is one of the valuable contributions of this paper and is worthy of the interest and attention of engineers, operators, and research men concerned with the activated sludge process.

As stated by Mr. Regester, information obtained from the design of large plants such as those at Milwaukee, Chicago, and Indianapolis, has guided the design of subsequent works. However, difficulties have been encountered in the operation of many of the smaller plants, and even in some of the larger plants, which have attempted to lean heavily on information gained at some other large city. This indicates strongly that the problems of each city are different and require "custom-made" rather than "ready-made" plants. The writers agree with the statement that "The need for additional study and research is evident." A fuller understanding of the basic principles of the process is of vital importance if the design of activated sludge plants is to be placed on a rational basis.

A factor not mentioned by the author that the writers believe to be an important design consideration is the effect of temperature on the rate at which activated sludges use oxygen. An extensive laboratory investigation of this factor has been made and the data presented elsewhere by the writers.[9] The change in temperature which occurs in most activated sludge plants in temperate climates between the extremes of late summer and late winter is usually in excess of 10° C. Simultaneous studies made on identical mixtures of activated sludge and sewage maintained at 10°, 15°, 20°, and 25° C reveal that the highest rates of oxygen usage are obtained under summer conditions (25° C) and the lowest rates under winter conditions (10° C). The most rapid stabilization is accomplished under summer conditions, and thus a much shorter period of treatment suffices at the higher temperatures for the sewage-sludge mixture to reach the base rate of oxygen utilization, which is indicative of stabilization of the organic matter in the sewage. At the lower temperatures the maximum rate of oxygen usage is much lower in magnitude, and therefore is prolonged for a greater period of time. Thus a longer detention period in the aerators is required under winter conditions to accomplish the same degree of sewage oxidation or stabilization. For plants operating with fairly uniform concentrations of aeration solids throughout the year, the distribution of air should be varied with the changing temperature to fit the varying oxygen-utilization rate curve, if most efficient plant operation is to be obtained. In order to take full advantage of this variable a plant must be equipped with extremely flexible aeration facilities.

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[7] Instr., Civ. Eng., Carnegie Inst. of Technology, Pittsburgh, Pa., on leave; Hydr. Laboratory, Univ. of Wisconsin, Madison, Wis.

[8] Post Doctorate Fellow, Hydr. and San. Eng., Univ. of Wisconsin, Madison, Wis.

[9] "The Influence of Temperature Upon the Rate of Oxygen Utilization by Activated Sludges," by C. N. Sawyer and G. A. Rohlich, *Sewage Works Journal*, Vol. XI, No. 6, November, 1939, p. 946.

The investigation previously reported[9] covered a series of studies made on activated sludges obtained from four cities and having flows varying from 450,000 gal per day to 120,000,000 gal per day. The 5-day B. O. D. strengths of the sewages being treated by these activated sludges varied from 140 ppm to 530 ppm. The results showed that the rate of oxygen usage at 20°, 15°, and 10° C was approximately 72%, 45%, and 25%, respectively, of the rate shown at 25° C, and the small divergence in the results obtained on the different sludges led to the proposal of the equation $y = 0.71\,x^{1.54}$, in which y is the relative activity expressed as a percentage of that at 25° C and x is the temperature in degrees centigrade.[10] If the over-all amount of oxygen used is a measure of the amount of biochemical oxygen demand stabilized, the aeration period during the cold winter months should be approximately three to four times as long as in the hottest part of the summer in order to attain an equal degree of stabilization of the activated sludge.

This is only one of the effects of temperature on the activated sludge process that the engineer must acquaint himself with if his design is to be based on fundamental knowledge, rather than on what is the accepted practice. Other factors worthy of consideration are the effect of temperature on (1) the rate of growth, (2) the volatile solids content, and (3) the nitrogen content of the sludges. These factors will be of importance in determining digester capacity, gas production for power purposes, and also in the design capacity of units used in those cases where filtration and drying of the waste activated sludge, for incineration or fertilizer use, is practiced.

Mr. Regester's statement that (see heading "Operation and Control of Process: Suspended Solids in Mixed Liquor") "The optimum suspended-solid content of the mixed liquor in the aeration tanks is best determined by actual plant performance" substantiates the seventh item in his stimulating list of phases of the activated sludge process requiring needed study and research. This item states the need for investigation showing the possible general relation between the optimum suspended-solids content of aerated mixtures and the character of the settled sewage. To gain a concept of the significance of variation of suspended solids on the oxygen requirements of activated sludges, studies[11] in 1938, have shown that, as the concentration of activated sludge increases, the rate of oxygen utilization also increases in nearly direct proportion to the concentration of sludge used. With high rates of oxygen usage produced by high sludge concentrations, the break in the rate curve,[12] indicating the completion of a certain phase of the oxidation, occurs most rapidly and the rate drops to a low level very quickly. This means that with high sludge concentrations the degree of stabilization needed for satisfactory treatment of the sewage is attained much sooner and that a shorter aeration period is sufficient, whereas a longer aeration period is required in the case of the slower rates of oxidation given by low suspended-solids concentrations to obtain an equal degree of stabilization.

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[10] "The Influence of Temperature Upon the Rate of Oxygen Utilization by Activated Sludges," by C. N. Sawyer and G. A. Rohlich, *Sewage Works Journal*, Vol. XI, No. 6, November, 1939, Fig. 3, p. 960.

[11] "Effect of Sludge Concentration and Temperature Upon Oxygen Utilization," by C. N. Sawyer and M. Starr Nichols, *loc. cit.*, Vol. XI, No. 1, January, 1939, p. 51.

[12] "Effect of Sludge Concentration and Temperature Upon Oxygen Utilization," by C. N. Sawyer and M. Starr Nichols, *loc. cit.*, Vol. XI, No. 1, January, 1939, Graph 4, p. 57.

It is interesting that the effect of increasing the concentration of aeration solids gives oxygen-usage results which are comparable with those obtained by increasing the temperature when the solids concentration remains constant. The long period of treatment required for stabilizing the sewage at low temperatures can be shortened materially by increasing the aeration solids during such periods. Thus, by varying the sludge concentration with the season or temperature, the degree of stabilization necessary can be accomplished under all conditions of temperature during the normal aeration period provided. To the knowledge of the writers, no activated sludge plants are operated on this basis at present. With the sludge concentration being varied as the temperature changes, the oxygen-utilization characteristics of the aeration mixture will remain much more uniform and less flexibility will be required in the aeration facilities. The application of these principles may prove valuable to some existing plants which have very little flexibility in their aeration devices.

The need for basic information on the optimum nutritional requirements of activated sludge and on the behavior of sludges fed on unbalanced diets cannot be stressed too greatly. Any engineer who has had a hand in the design and operation of a sewage treatment plant for a small city of less than 10,000 population realizes that the problems to be solved in the small city are just as numerous as, and perhaps more critical than, those in a large city. The influence of specific wastes in sewage on the ultimate behavior and operation of the treatment plant is of concern to all. Such fundamental information as can be obtained from basic nutritional studies on activated sludge should be of considerable value in determining the applicability of the activated sludge process to the disposal of industrial wastes that may lend themselves to biological treatment.

The current trend of the federal government toward entering the field of stream-pollution control is certain to tax the present limited knowledge of sewage and waste disposal to the utmost; and, with the author's thought-provoking list of items needing further study and research, the engineer has before him a challenge that he will have to meet not only on engineering ground but on the fields of allied subjects such as chemistry, biochemistry, and bacteriology as well.

E. SHERMAN CHASE,[13] M. AM. SOC. C. E. (by letter).—A compilation of data relating to the design of thirty sewage treatment plants must have been a difficult and laborious task, and the author deserves the gratitude of sanitary engineers for his presentation of these data.

The non-uniform character of sewage, and the variable conditions under which its treatment occurs, make it necessary to use caution in the application of the results of laboratory research to practical problems of design and operation. For half a century the art of sewage treatment has been subjected to scientific investigation, and in spite of the mass of scientific research which has been conducted, sewage treatment continues to remain an art in which experience and judgment have at least as much weight as the researches of the laboratory. Therefore, design data such as presented by the author are of genuine value as an aid to the judgment of other engineers. However, such data need to be supplemented by the results of operation as given for nine of the thirty plants discussed by the author.

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[13] Cons. Engr. (Metcalf & Eddy), Boston, Mass.

The increase in activated sludge plant capacity in the United States appears to be due mainly to the construction of a few large plants such as those at Chicago, New York, Cleveland, and Columbus. On the other hand, it is true that numerous smaller activated sludge plants have been built in recent years. Generally this type of plant is less practical for the small municipalities than for the larger cities, due to the complexity of the process and the need of relatively expert supervision. The smallest installation given in the author's tabulation is that at Ann Arbor, Mich., with a capacity of 4.5 mgd. Under certain local conditions activated sludge plants of even smaller capacity are sometimes more suitable than any other type, although it is usually advisable to install the simplest type of treatment for the small municipalities.

At Leominster, Mass., where local conditions made it inadvisable to install an Imhoff tank-trickling filter plant, a small activated sludge plant has been in operation since 1938. The basic design data for this plant, comparable with the design data given in Table 1, are as follows:

Column No.	Description	Value
1	Location, Leominster, Mass.	....
	Nominal Capacity:	
2	Sewage flow, in million gallons daily	2.0
3	Human population, in thousands	25.0
	Year of Starting:	
4	Construction (WPA)	1934
5	Operation—preliminary treatment	1937
	activated sludge treatment	1938
6	Type of sewerage system	separate
	Preliminary Treatment:	
7	Coarse screen openings (cleaned mechanically), in inches	1
8	Grit chamber	yes
9	Pre-aeration	no
10	Number of mechanically cleaned settling tanks	2
11	Sedimentation period, in hours	1.5
	Aeration:	
12	Mixed liquor detention period, in hours	5.5
13	Returned sludge (percentage)	25

Column No.	Description	Value
	Final Sedimentation, Mixed Liquor:	
14	Settling rate, in gallons per square foot daily	800
15	Detention period, in hours	2.2
16	Further treatment	none

From this tabulation it will be seen that the Leominster plant is more or less comparable, as regards design data, with the plants listed in the paper. The sludge is digested in a separate digestion tank and dewatered on open sludge beds. Since its initial operation the Leominster plant has been operated successfully with no more than the usual difficulties encountered in starting a new plant. The removal of biochemical oxygen demand and suspended solids has averaged about 90%. At times bulking has occurred in the final settling tank, which appears to take place when the strength of the sewage is greatest and to disappear when the sewage is weak. Bulking continues to remain something of a mystery and at Leominster the sludge index is not a wholly reliable criterion as to whether or not bulking is likely to occur.

There is some question in the writer's mind as to whether the biochemical oxygen demand test alone should be relied upon to show the degree of purification effected. For example, at Leominster the 1940 tests made by the State Department of Public Health showed an oxygen demand removal of 89.2% as compared with removals of 72.3% and 71.2% for albuminoid nitrogen consumed. The fact that there is more than one stage for oxygen demand indicates that perhaps the 5-day oxygen demand removals do not tell the entire story and that some of the older tests are still of value as indexes of purification.

Certain innovations suggested by the author, such as tapered aeration and the use of magnetite filters for polishing activated sludge effluents, seem to be, as yet, of unproved value. As regards aeration, it is evident that it serves two purposes: First, agitation of the mixed liquor; and second, the supplying of oxygen. The amount of oxygen present in the air applied, in quantities sufficient to provide agitation, is more than enough to maintain aerobic conditions in aeration tanks under ordinary conditions. The problem of aeration involves not so much the matter of adequacy of supply as it does of getting the air into the mixture sufficiently rapidly. From the theoretical standpoint it would appear somewhat more logical to taper the loading rather than to taper the air, particularly as the time element is an important factor as regards oxygen absorption.

In conclusion, the writer wishes to emphasize his personal conviction that the designer should consult the operating staffs of treatment plants at every opportunity in order that his designs may embody the benefit of practical operating experience.

ROBERT T. REGESTER,[14] M. AM. SOC. C. E. (by letter).—The discussions of the writer's paper have contributed valuable supplementary information regarding activated sludge practice.

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[14] Cons. Engr., Baltimore, Md.

The comprehensive scope of the subject necessarily restricted its presentation by the writer to concise statements of fact with occasional interpretative comments. The effort was made to present an impartial summary of American practice since 1930. That period was chosen because it represented an outstanding recent era of advancement in design, expansion in plant construction, improvement of operation, and intensification of research activities. Both agreements and departures in design and operating practices, regardless of the respective merits, were noted in the paper. Advocacy of particular methods and innovations was avoided. Undue emphasis upon the developments in any particular locality was undesirable. The writer's aim was to stimulate the interest of sanitary engineers in the problems of activated sludge practice and to encourage their efforts toward its improvement and standardization.

It appears that Mr. Pearse may have viewed the paper primarily in the light of Chicago's practice and without regard to the writer's objectives. An appraisal was purposely omitted from the paper as the writer preferred a critical analysis and valuation of developments by the reader. Mr. Pearse elaborates upon certain aspects of activated sludge practice in the Sanitary District of Chicago and upon features of its plants. Also, he notes important pioneer contributions by the Sanitary District.

In referring to the North Side Works, he mentions that the average capacity of the plant was rated in 1928 at 180 mgd. For purposes of accurate record, it should be noted that this original rating (3) was 175 mgd (item 3, Table 1). Parts of the works which could not be readily extended were designed for 1960 conditions.[15] However, it has been explained (3) that, because of the unusually rapid population growth of the North Side area during the 1920–1930 decade, the plant started with an average sewage flow of about 200 mgd, which was considerably in excess of its design capacity. Furthermore, the plant successfully carried the load. The original liberally designed aeration period and the foresighted hydraulic provisions made possible the newly adopted rating of 250 mgd with the addition of only final settling tanks. The twelve units added in 1937 (item 3, Table 4) represented an increase of 30% in the surface area of the plant's final settling tanks. On the other hand, the rated capacity of the plant was increased 43%. Therefore, on the basis of 20% return sludge, the mixed-liquor settling rate was increased from the original design basis of 1,180 gal per sq ft daily to 1,300 gal per sq ft daily for the rating of 250 mgd. Likewise, the mixed-liquor detention period in the aeration tanks was decreased from the original design basis of 6.3 hr to 4.4 hr. These supplementary data may give a clearer understanding of attending conditions under the present rating. Their inclusion in Table 1 would have unbalanced the comparative design data and possibly would have obscured the recent tendency toward shorter aeration periods as noted in the paper.

Mr. Pearse questions the correctness of the writer's description of the method of sludge handling adopted by the Sanitary District as a "use of waste heat." It is noted by Mr. Pearse that the heat recovered by burning the sludge (when burned) is used in drying the wet sludge cake and that the generation of steam depends primarily on the use of additional coal. This implies that at least some of the heat is also used for the generation of steam.

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[15] "Engineering Works," *Booklet*, The Sanitary District of Chicago, August, 1928, p. 71.

The following editorial comment[16] upon the new Southwest plant appears to share the writer's classification of the method:

> "By far the most significant feature, however, is the plan to use a sludge-incineration process for waste-heat utilization in conjunction with a steam-generating unit that will operate pumps, blowers and house generators. Disposal of sludge by dewatering and incineration as contemplated at Chicago seems at present to be the most promising solution of a troublesome problem, particularly when large volumes must be dealt with."

The following statement is also enlightening:[17]

> "The method of dewatering and incineration adopted by the Sanitary District has the advantage of year-round operation, requires less area and is lower in first cost than the more conventional method, thereby producing large savings in the total cost of the works. Lower operating costs are expected also. Further, by burning the sludge under the boilers which generate steam for plant power some of the heat value can be recovered from the sludge. Also dried activated sludge can be withdrawn for sale as fertilizer as desired."

Further explanation is given as follows:[18]

> "Dried sludge averages 7,000 Btu per pound which is enough heat to evaporate the moisture in the sludge when the filters are operated to produce a filter cake with 80% moisture. At lower moisture content there will be some heat available for generating steam and at higher content some heat will have to be supplied by burning more coal.
>
> "Steam for plant operation will be generated by four water-tube boilers, each having maximum capacity to produce 110,000 lb of steam per hour at 425 lb gauge pressure and 735 degrees final temperature. Pulverized coal and dried sludge will be used for fuel."

The writer hopes that the foregoing quotations have answered in his favor the question which was raised. The real question seems to be how much power will be generated by the excess heat from the sludge when it is burned under the same boilers with additional pulverized coal. It is regretted that Mr. Pearse did not include the design heat-balance diagram for the plant in his discussion.

With reference to preliminary treatment, Mr. Pearse comments that the use of fine screens should have been noted in the paper. The existing installations of fine screens at Milwaukee, Indianapolis, Pasadena, and Charlotte (items 4, 7, 18, and 29) were noted in the footnotes of Table 1. Fine screens were not specifically discussed because to the writer's knowledge they have not been installed in any activated sludge plants, of large and medium capacities, constructed in the United States since 1930. However, in some instances consideration was given to the use of fine screens,[19] but they were rejected in favor of the more effective preliminary settling tanks.

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[16] *Engineering News-Record*, Vol. 115, No. 6, August 8, 1935, p. 202.

[17] "Budget Report on Engineering Work—1938, The Sanitary District of Chicago," by William H. Trinkaus, M. Am. Soc. C. E., chief engineer, p. 11.

[18] "Southwest Treatment Works of the City of Chicago," by William H. Trinkaus, *Municipal Sanitation*, Vol. 10, No. 4, April, 1939, p. 231.

Mr. Pearse confirms the writer's personal views with regard to the desirability of providing preliminary sedimentation. In addition, Mr. Pearse has offered an excellent reason for including preliminary settling in the Calumet and Southwest works which was not mentioned in the paper—that is, a mixture of fresh and activated sludge will dewater more readily on a vacuum filter than activated sludge alone, as shown by Chicago tests.

Both Mr. Pearse and Mr. Roe have noted the absence in the paper of a specific discussion relating to porous diffusers, except under "Needed Study and Research." The omission of that topic was intentional as diffusers had been recently discussed before the Society by S. W. Freese, M. Am. Soc. C. E. (12), who noted the related problems and trends.

The writer mentioned that blow-off valves, for either the air discharge manifold or the discharge pipe of each blower, were a new feature of centrifugal blower installations. Mr. Pearse stated that such use for blow-offs is almost unknown, and indicated that a blow-off for rotary positive-displacement blowers may be highly desirable. The writer was not referring to the pressure-relief valves as commonly used with rotary blowers. On the contrary, he mentioned that blow-off provisions are useful in minimizing the "pumping effect" when a centrifugal blower is being added to those already in operation. The necessity for this provision is stressed by at least one of the four prominent manufacturers of centrifugal blowers. Another of these firms, which has furnished many of the blowers for the Sanitary District, having straight, radial-blade impellers, finds it unnecessary to have a blow-off valve. At Columbus, provision was made in the design of the blower piping for adding a blow-off valve on the discharge pipe of each blower to satisfy the manufacturer first mentioned, in the event that his units should be obtained as the result of competitive bidding. Actually, blowers of the straight, radial-blade type were installed. However, a blow-off from the discharge header will permit the system pressure to be lowered by exhausting to atmosphere, when desired. In the Cleveland (Easterly) and Baltimore (Back River) plants (items 6 and 9, Table 3), centrifugal blowers were furnished by the same manufacturer who desired the blow-off provision. In the latter case, the writer knows that a blow-off valve was installed for each unit at the insistence of the manufacturer. Apparently, the inherent pressure-capacity characteristics for a particular blower design dictate the necessity for blow-off valves so as to avoid the severe disturbances in the connected system when a blower is started and passes through its inherent unstable zone of operation (at about 50% of rated capacity) where "pumping" or "surging" occurs.

The writer was aware of some of the final sedimentation tests referred to by Mr. Pearse, and had studied the results. It was hoped that part of these data might appear as published contributions to the knowledge pertaining to activated sludge practice. The writer recommended further investigation of the efficiency of final settling tanks and factors affecting their design, and he continues to stress the need for such investigations.

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[19] See, also, report, "Proposed Sewage Treatment Plant at Ward's Island," by George W. Fuller, consulting engineer, Board of Estimate and Apportionment, City of New York, 1928, p. 88. See also reference (8) in the Appendix of the paper.

Since the paper was presented, test data on concentrating activated sludge with a continuous feed centrifuge, of the type as installed at Columbus and Lansing, have been published.[20] These tests were conducted at Peoria, and showed that the centrifuge is capable of increasing the concentration of waste activated sludge from 1% to 5% solids. For that range of concentration, the total cost was estimated at $5.79 per ton of dry solids. In connection with the design of the Baltimore plant, the writer observed carefully conducted tests by personnel of the City of Baltimore (in cooperation with the manufacturer) of an experimental unit at Lima in 1937. Certain mechanical difficulties were troublesome at that time, but it was expected that these might be remedied later. However, as a result of these tests it was decided not to install centrifuges for activated sludge concentration at the Back River plant (item 9, Table 5).

Mr. Pearse also refers to the writer's statement regarding the probability that increased depth of sludge blanket produces a greater density of sludge. It is unfortunate that the 1931 test data of the Sanitary District, mentioned in the discussion, were not included. The writer had in mind a relatively small variation in the density of activated sludge with increased depth. He has viewed the results of certain tests which had shown some increase in the density for sludge depths up to 10 ft. The value of such slight increases in sludge density can be appreciated by referring to Fig. 3. If the solid content of the sludge is increased from 1.0% to only 1.5%, it will be noted from Fig. 3 that the rate of sludge returned (to maintain 2,000 ppm of suspended solids in the mixed liquor) can be decreased from 25% to 15% of the corresponding sewage flow with consequent saving in return-sludge pumping costs. The statement did not refer to the concentration of activated sludge by either thickening tanks or centrifuges to perhaps 4 or 5% solids.

Besides discussing the interesting developments in the evolution of porous diffusers, Mr. Roe mentions the recent use of diffuser tubes in preference to plates for the activated sludge plant recently constructed at Gary, Ind.[21] For that 40-mgd plant, the designers selected stationary tubes, each 3 in. internal diameter by 24 in. long, which are set 24 in. above the floor of the aeration tanks. They are staggered along the submerged air header to give a spacing of 1 ft 4 in. A permeability rating of 40 was used. This development in diffuser practice for spiral-flow tanks in a large plant is significant in view of the fact that plates have given satisfactory service in numerous plants for relatively long periods of time.

Messrs. Rohlich and Sawyer discuss their findings from a praiseworthy investigation of the influence of temperature upon the rate of oxygen utilization by activated sludges. They also discuss the combined effect of suspended-solids content and temperature upon oxygen utilization as shown by their studies. It is hoped that the relationships which they express will be tried and observed in plant operation. Basic research of that character should aid materially in improving activated sludge practice, and should be encouraged.

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[20] "Concentrating Activated Sludge with a Continuous Feed Centrifuge," by L. S. Kraus and J. R. Longley, *Sewage Works Journal*, Vol. 11, No. 1, January, 1939, p. 9.

[21] "Sewage Treatment at Gary," by L. R. Howson, M. Am. Soc. C. E., *loc. cit.*, Vol. 11, No. 6, November, 1939, p. 994.

Mr. Chase wisely refers to the caution that is necessary in the application of the results of laboratory research to the practical problems of design and operation. The conclusions from small-scale experiments, often conducted under artificial conditions, may be misleading unless proved under conditions of normal plant operation. In presenting the suggestions for needed study and research, the writer endeavored to interest plant operators, as well as research workers, in the possibilities for the further investigation of certain phases of activated sludge practice.

The basic design data for the 2.0-mgd plant at Leominster, as presented by Mr. Chase, is a valuable adjunct to the data given in the paper. The writer is pleased that a small activated sludge plant received deserving attention in the discussion.

Mr. Chase raises an important question when he suggests that some of the older tests may also be valuable indexes of the degree of purification obtained in activated sludge treatment. This matter should receive the attention particularly of plant operators and chemists.

In conclusion, the writer is gratified to learn from the discussions and from letters which he has received that the data presented in the paper may be found useful. He extends his sincere appreciation to all who have contributed to the discussions.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 2103

TRANSIENT FLOOD PEAKS

By Henry B. Lynch,[1] M. Am. Soc. C. E.

With Discussion by Messrs. Ivan E. Houk, Gordon R. Williams, Donald M. Baker, James M. Fox, F. C. Finkle, A. L. Sonderegger, Harold C. Troxell and R. Stanley Lord, Karl J. Bermel, R. W. Davenport, Walter J. Wood and Maxwell F. Burke, Franklin Thomas, and Henry B. Lynch.

Synopsis

Floods of the so-called "cloudburst" type yield momentary runoff peaks entirely out of proportion to the rate of rainfall. They are caused by an abrupt increase in rainfall and runoff. Their magnitude is controlled by many factors, of which probably the most important are the rate of increase and the intensity of the rainfall. One of the most noteworthy examples of this type of flood was the flood near Los Angeles, Calif., on the morning of January 1, 1934.

The more detailed study from which this paper was prepared has been placed on file for reference at Engineering Societies Library.[2]

General

An abrupt increase of rainfall, under some conditions, produces a transient runoff peak from a small area exceeding by many fold the flow which could be sustained by the rainfall upon the area. In computing flood flows it is usual to consider that the peak discharge is at some rate less than the rainfall on the area, and that runoff never exceeds rainfall except in the case of a temporary damming of the channel. This is true of the runoff from a rain of uniform intensity, and it is true also of the sustained runoff in all cases. It is sometimes entirely untrue of the momentary maximum runoff when the rainfall is increasing suddenly. With a runoff that is growing greater there is a tendency for the deeper, faster moving water from upstream to overtake the water in front of it. When the increase is abrupt there may accumulate, at the junction of the fast and slow moving waters, a steep-fronted surge popularly termed a "wall of water" many fold larger than the sustained runoff behind it. The same effect

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Note.—Published in November, 1939, *Proceedings*.

[1] Cons. Engr., Los Angeles, Calif.

[2] 33 West 39th Street, New York, N. Y.

occurs, under conditions favoring it, at the beginning of an exceedingly abrupt and heavy rain. In large areas the runoff seldom increases rapidly enough for these surges to occur.

Under usual conditions, where rain falls steadily over the entire area of the canyon, the depth and quantity of runoff increases continuously from the head of the canyon to its mouth. However, when an abrupt, heavy rain begins near the head of the canyon, under circumstances such as to favor a rapid runoff, a reversal of normal runoff conditions may exist temporarily in the upper part of the canyon. Under such conditions, the water upstream may be deeper than the water in front of it, for a short time, and runoff may take some such form as is shown in Fig. 1. Abrupt rainfall, concentrated near the head of the canyon, is not uncommon. It is the most favorable condition for the formation of the surge. Under conditions favoring it, the surge builds up to a size, and to a destructive power, which seem entirely out of proportion to the rate of rainfall.

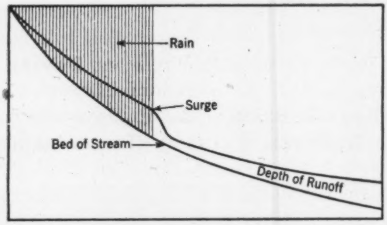
Rain
Surge
Bed of Stream
Depth of Runoff

Fig. 1.—Abnormal Runoff from Rain at Head of Canyon

An increase of rainfall near the head of the canyon is not the only cause that can produce this reversal of normal runoff conditions. The topography of the canyons in Los Angeles County (see Figs. 2 and 3) which emitted surges on January 1, 1934, is such that an abrupt increase of rainfall covering the entire canyon might cause a surge almost as violent as would be the case where the rainfall increase occurred only near the head of the canyon.

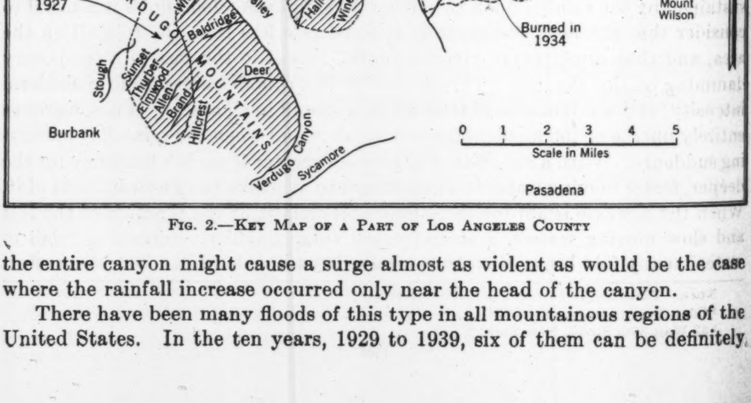
Burned in 1933
Mount Sister Elsie
Haines
Blanchard
Cooks
Dunsmuir
Eagle
Shields
Goss
Pickens
Snover
Hall-Beckley
Winery
SIERRA MADRE MOUNTAINS
Arroyo Seco
Fern
Burned in 1934
Mount Wilson
N
Tuna
Burned in 1927
VERDUGO MOUNTAINS
La Canada Valley
Whiting
Baldridge
Deer
Stough
Sunset
Thurber
Elmwood
Allen
Brand
Hillcrest
Verdugo Canyon
Sycamore
Burbank
0 1 2 3 4 5
Scale in Miles
Pasadena

Fig. 2.—Key Map of a Part of Los Angeles County

There have been many floods of this type in all mountainous regions of the United States. In the ten years, 1929 to 1939, six of them can be definitely identified as having occurred in Southern California. In all but one of these cases, the floods were confined to a small area, with no loss of life, and comparatively small property damage.

All the canyons from which these surges emerged were upstream from thickly settled suburbs. In several cases automatic rain gages were located within the canyons which generated the surges, and in each of the other cases automatic gages were in operation nearby.

Flood of January 1, 1934

The most destructive and extreme examples occurred in the mountains in Los Angeles County, California, on January 1, 1934. During the entire day and evening of December 31 a heavy rain had fallen, coming from the south, and thoroughly soaking the ground. Before evening, the thin soil covering the steep mountains had attained a high degree of saturation, and runoffs were large. Just before midnight the rain suddenly increased for a short time from a rate of about 0.5 in. to about 1.4 in. per hr across a five-mile front which included parts of the Sierra Madre and of the Verdugo Mountains to the south. The same thing occurred, but to a lesser extent, in other parts of the county.

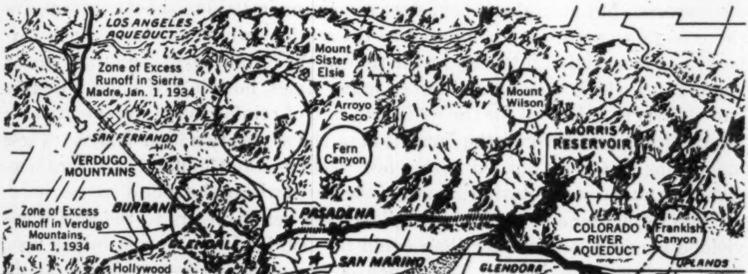
LOS ANGELES AQUEDUCT
Mount Sister Elsie
Zone of Excess Runoff in Sierra Madre, Jan. 1, 1934
Arroyo Seco
Mount Wilson
SAN FERNANDO
Fern Canyon
MORRIS RESERVOIR
VERDUGO MOUNTAINS
Zone of Excess Runoff in Verdugo Mountains, Jan. 1, 1934
BURBANK
GLENDALE
PASADENA
COLORADO RIVER AQUEDUCT
Frankish Canyon
SAN MARINO
GLENDORA
UPLANDS
BEVERLY HILLS
Hollywood
Sycamore Canyon
POMONA
ONTARIO
LOS ANGELES
RIVER
RIO HONDO
WHITTIER
SAN GABRIEL
RIVER
COUNTY
COUNTY
RIVER
SANTA ANA
COMPTON
FULLERTON
TORRANCE
CORONA
ANAHEIM
LOS ANGELES
LOS ANGELES
ORANGE
LONG BEACH
SANTA ANA
PACIFIC OCEAN

Fig. 3

The response of the runoff was prompt and violent. Within ten minutes before and after midnight surges of water, heavily laden with debris, emerged from each of the steeper canyons. These surges were many fold larger than could have been sustained by the rainfall upon the area. None of the canyons drains an area as great as 2 sq miles, and only three have areas greater than 1 mile. The surges emerged first from the canyons to the west. In Sierra Madre, the surge from Hall-Beckley Canyon seems to have been about twenty minutes later than that from Haines Canyon. The magnitude and violence of these surges are difficult to describe or picture adequately. Some of the canyons in the Sierra Madre showed flow sections which would seem incredible if they were not amply established by numerous photographs, measurements, and observations. In the period since this flood most of the high water marks have disappeared, but enough of them were remaining in 1939 to show how excessive these sections were when compared to the sections which would be usually expected. The rainfall intensity at the time of the formation of the surges was far below rates which have been recorded many times, both in Los Angeles County and elsewhere. Except for the moderate rate of rainfall, all of the conditions in the Sierra Madre favored the formation of the surges to a high degree. These mountains had been burned over a few weeks before the flood and were completely denuded of vegetation in most of their area within the zone of excess runoff.

The term "zone of excess runoff," as used herein, refers to the area which yielded runoff peaks greater than could be sustained by the rain falling upon the area (see areas in Fig. 2). Much of the area referred to herein as the "normal zone" showed rainfalls of similar amount and characteristics, but one or more of the factors necessary to the formation of the surges at such moderate rainfall rates were lacking.

These surges were confined to the zone of greatly increased rainfall on the south slope of the Sierra Madre and in the eastern portion of the Verdugo Mountains. They covered a roughly triangular area 5 miles on a side. The western part of the Verdugo Mountains gave normal runoffs, as did also Sycamore Canyon, about 1 mile east of these mountains. The surges showed a definite gradation from the great heights in the recently burned areas to the normal runoffs in the unburned areas outside of the zone of excess runoff. The greatest surges were those in the then recently burned area. Then followed in order, with some overlapping, the surges in the 1927 burn in the Verdugo Mountains, within the zone of excess runoff; the surges in the unburned areas in the zone of excess runoff; and the runoffs both in the burned and unburned areas outside of the zone of excess runoff.

Flow sections were measured in thirty-one places, and included twenty-two canyons or branches in the zone of excess runoff. In Table 1 the canyons have been segregated into the foregoing classifications. The unit used is the cross section of the flow for each square mile of catchment area. No attempt has been made to compute the peak flows.

In the zone of normal runoff, velocities and discharges have been computed by the Manning formula for determining the flow in open channels:

$$V = \frac{1.486}{n} R^{0.67} S^{0.50} \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

TABLE 1.—HYDRAULIC PROPERTIES OF SECTIONS INVOLVED IN THE FLOOD OF JANUARY 1, 1934

Sections	Drainage area (see Figs. 2 and 3)	WITHOUT CHECK DAMS						WITH CHECK DAMS			Cross-sectional area per square mile of catchment
		Area of cross section A, in square feet	Feet		Coefficient of roughness, n†	Hydraulic slope, S	Catchment area, in square miles	Area of cross section A, in square feet	Feet		
			Wetted perimeter, P	Hydraulic radius, R					Wetted perimeter, P	Hydraulic radius, R	
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(2)	(3)	(4)	(10)
	(a) AREAS BURNED IN 1933										
A	Winery	26	29	0.9	0.030	....	...	....	...	...	....
		130	29	4.5	0.040	0.080	0.14	....	...	...	1,114
B	Hall-Beckley	813	101	8.0	0.040	0.069	0.72	680	99	6.9	944
C	Pickens:										
C-1	Above Mullaly	456	61	7.5	0.025	0.100	1.28	....‡	...‡	...‡	....
C-2	Mullaly Fork	536	59	9.1	0.035	0.077	0.35	493	57	8.7	1,408
C-3	Above Gould's Castle	711	81	8.8	0.050	0.087	1.78	621	73	8.5	349
C-4	Gould's Castle	521	71	7.3	0.050	0.090	1.78	406	66	6.1	222
C-5	Mountain Street	693	80	8.7	0.035	0.072	1.80	590	77	7.7	328
C-6	Cross Street*	1,231	298	4.1	0.045	0.067	1.81	1,166	292	4.0	644
D	Goss	64	24	2.7	0.035	0.074	0.28	....	...	...	229
E	Eagle:										
E-1	East Fork	66	23	2.9	0.055	0.133	0.06	26	18	1.4	433
E-2	West Fork	154	33	4.7	0.055	0.164	0.16	....	...	...	962
F	Shields	296	46	6.4	0.050	0.121	0.24	212	41	5.2	883
G	Dunsmuir	1.134	214	5.3	0.050	0.104	0.86	1,016	210	4.8	1,181
H	Cooks	625	77	8.1	0.035	0.088	0.65	473	74	6.4	728
I	Blanchard	305	72	4.2	0.035	0.061	0.45	....	...	...	677
J	Haines	299	49	6.1	0.040	0.122	1.20	....	...	...	239
	Average	....	...	...	....	....	...	....	...	...	689
	(b) BURNED IN 1927; ZONES OF EXCESS RUNOFF										
K	Brand:										
K-1	Total flow	80	22	3.6	0.014	....	...	....	...	...	....
		153	90	1.7	0.030	0.060	1.06	....	...	...	220
K-2	East Fork	180	38	4.7	0.055	0.089	0.43	....	...	...	418
L	Allen:										
L-1	Total flow	103	79	1.3	0.030	0.037	0.34	....	...	...	303
L-2	West Fork	83	27	3.1	0.035	0.059	0.22	....	...	...	377
M	Thurber	10	9	1.1	0.030	0.084	0.03	....	...	...	333
N	Elmwood	15	25	0.6	0.015	0.049	0.33	....	...	...	45
	Average	....	...	...	....	....	...	....	...	...	283
	(c) UNBURNED CANYONS; ZONES OF EXCESS RUNOFF										
O	Whiting	57	20	2.8	0.040	....	...	....	...	...	....
		19	11	1.7	0.055	0.034	0.84	....	...	...	90
P	Baldridge	116	38	3.0	0.035	0.054	0.73	....	...	...	159
Q	Deer	90	28	3.2	0.040	0.100	0.41	....	...	...	219
R	Hillcrest	66	42	1.6	0.015	....	...	....	...	...	227
		18	22	0.8	0.025	0.065	0.37	....	...	...	....
	Average	....	...	...	....	....	...	....	...	...	174
	(d) UNBURNED CANYONS; NORMAL ZONES										
S	Arroyo Seco*	112	100	1.1	0.050	0.032	13.40	....	...	...	8.3
T	Sycamore	42	37	1.1	0.015	0.034	1.80	....	...	...	23.3
U	Stough:										
U-1	North Fork	9	10	0.9	0.030	0.035	0.84	....	...	...	10.7
U-2	East Fork*	8	12	0.7	0.035	0.030	0.58	....	...	...	13.8
V	Tuna	22	18	1.2	0.025	0.016	5.51	....	...	...	4.0
	Average	....	...	...	....	....	...	....	...	...	12.0

* Approximate. † Roughness of channel only. No attempt has been made to compensate for reduced velocities due to debris load. ‡ Values not known.

in which V = velocity in feet per second; n = coefficient of roughness; R = hydraulic radius; and S = hydraulic slope. No conditions are known which would affect its use in these cases. Five of the sections were measured in the zone of normal runoff, as reported in Table 2. All of these measurements were made in canyons which were contiguous to canyons in the zone of excess runoff. The properties of these sections are shown in Table 1(*d*).

TABLE 2.—VELOCITIES AND DISCHARGES COMPILED BY THE MANNING FORMULA; UNBURNED CANYONS; NORMAL ZONE

Sections (see Table 1(*d*))	Drainage area (see Figs. 2 and 3)	Velocity, V, in feet per second	Rate of flow, Q, in cubic feet per second
S	Arroyo Seco	5.7	638
T	Sycamore	23.0	966
U	Stough:		
U-1	North Fork	9.1	82
U-2	East Fork	6.5	51
V	Tuna	8.6	189

The canyons in the normal zone gave runoffs averaging 161 cu ft per sec for each square mile of catchment area. These runoffs are not uncommon. They have been exceeded many times in all parts of the United States.[3] A comparison with them affords a rough idea of the great magnitude of the wave fronts in the zone of excess runoff. They also show what short distances separated canyons in the zone of excess runoff from canyons in the normal zone whose cover and gradients were similar.

The automatic rain gages in this vicinity showed no rainfalls lasting 15 min, at a rate as intense as 2 in. per hr. Even with a very high percentage of runoff this rainfall would not have sustained flow sections greater than from 30 to 40 sq ft per sq mile of catchment area.

The average of measurements made is as follows:

Description	Square feet of flow section per square mile of catchment
In the zone of normal runoff, flow sections averaged	12
In the unburned areas in the zone of excess runoff, flow sections averaged (or 15 times larger than average flows in the normal zone)	174
In the 1927 burn in the zone of excess runoff, flow sections averaged (or 23 times larger than average flows in the normal zone)	283
In the 1933 burn in the zone of excess runoff, after deducting as below for check dams, flow sections averaged (or 57 times larger than the average flows in the normal zone)	689

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[3] An excellent "Bibliography Relating to Flood Flow, Intense Rainfall, and Frequencies" is contained in U. S. Geological Survey *Water Supply Paper No. 771*, "Floods in the United States," by Clarence S. Jarvis, M. Am. Soc. C. E., and others, 1936, pp. 468–487.

A runoff at the rate of 1.56 in. in depth each hour over the entire catchment area is required to sustain a flow of 1,000 cu ft per sec for each square mile. A flow of 4,000 cu ft per sec per sq mile requires a runoff at the rate of more than 6 in. in depth each hour, and in these streams would have a section of approximately 130 sq ft for each square mile.

The flow sections in the zone of excess runoff are so large that great care was used to assure that they would be representative of the conditions at the time of the peaks. Many of these measurements were made at locations where there could be no fear that the section presented a distorted situation because of scouring of the bed after the highest elevation of the water surface had occurred.

In the mountains burned in 1933 the measurement in Haines Canyon was made at a concrete U. S. Geological Survey gaging station (see section J, Table 1). Another of the measurements was made on a check dam which remained intact (section E-2, Table 1).

In the zone of excess runoff in the Verdugo Mountains the section which measured the total flow of Brand Canyon was taken in a concrete storm channel. The water had spread into an orchard and some cutting of the ground had occurred, but not much (see section K-1, Table 1). The Allen Canyon flow was measured on an uninjured check dam (section L-1, Table 1). Two of the sections were measured on concrete streets (sections N and R, Table 1). In the zone of normal runoff one of the sections was measured on a concrete street (section T, Table 1). The other sections in this zone showed no signs of scouring.

As to four of the sections in the zone of excess runoff, it could not be said, from indications upon the ground, whether scouring of the bed had occurred after the maximum water elevation was reached (sections C-1, D, I, and P, Table 1). Scouring would be expected to show in the form of sections much larger than the average, but there is nothing to indicate such a situation. Each of the four sections is smaller than the average in its classification. One section showed definite signs of scouring, but gave no clue to the location of the bed of the channel before the flood. It has not been used in computing averages (section C-1, Table 1).

At sections C, E, F, G, and H, Table 1, the remains of small check dams made of wire-bound boulders were present close to the point of measurement. Before the flood these check dams, placed about 50 yd apart, were intact, the space above them largely filled with debris from the canyons. Most of them were swept away by the flood. It is not known definitely whether the check dams were destroyed before or at the time of the surges.

In the absence of information, it is assumed that check dams were intact at the moment of the maximum peak. This assumption necessitates the decrease of such measured sections to allow for the areas that these check dams occupied before their destruction. Remains of check dams showed the extent of scouring of the adjacent channels. At none of the measured sections (except perhaps section C-1, which was not used as a basis for averages) would the check dams, if intact, have reduced the measured area by more than the lowest 4 ft. This value has been used for the reduction where check dams were located sufficiently near so that they could possibly affect the measurement.

A comparison of the data in Table 1 shows that the average hydraulic properties of the channels in the various classifications tend slightly to equalize the values, rather than to accentuate the differences.

AREAS IN THE ZONE OF EXCESS BURNED IN 1933

Hall-Beckley Canyon.—This canyon lies in the burned part of the Sierra Madre near the eastern limit of the Verdugo Canyon basin. At a point in the canyon where the catchment area is 0.67 sq mile, the trees are still completely scoured of bark on their upstream side to a height of many feet above the stream bed. The appearance and shape of the channel at this point are shown in Fig. 4, taken more than three years after the flood. This view was taken at one of the few remaining places which still give an idea of the magnitude of these surges. The stream bed is 5 ft below the bank on which the tree is growing. About 2,000 ft downstream from this point a measurement was made in a straight section where marks of the water surface were well defined. The catchment area above this section measured 0.72 sq mile. The flow section was 95 ft wide at the surface and almost 13 ft deep to the bed of the channel. After deducting the lowest 4 ft for the area occupied by check dams before the storm, the flow section was 680 sq ft, or at the rate of 944 ft for each square mile of catchment area. The slope was 0.069. Indications of water surface were unmistakable. At the height adopted, the bark was completely scoured from the upstream face of an oak tree 9 ft from the left bank. Splash from the water extended from 10 to 12 ft higher into the tree, equivalent to the velocity

FIG. 4.—HEIGHT OF SCOUR IN HALL-BECKLEY CANYON

head at about 25 ft per sec. There could be no doubt of either the water surface or the high velocity.

Where the canyon joins the debris cone it has an area of 0.86 sq mile. At this point it is met by the channel from Snover and Weber canyons and 0.56 sq mile is added to the area. Below the confluence the flow enlarged the channel, which previously had been about 12 ft wide and 4 ft deep, until it was more than 50 ft wide and 6 ft deep. Notwithstanding this increase in area, the channel was so inadequate to carry the water that at its maximum flow the water was 5 ft deep in the yard of the residence shown in Fig. 5 and drowned two people there who had been clinging to a truck. This water included all of the flow from the Hall-Beckley Canyon, but only part of the flow from the Snover area. Testimony of many witnesses in court showed that the channel was of this size before the occurrence of the surge.

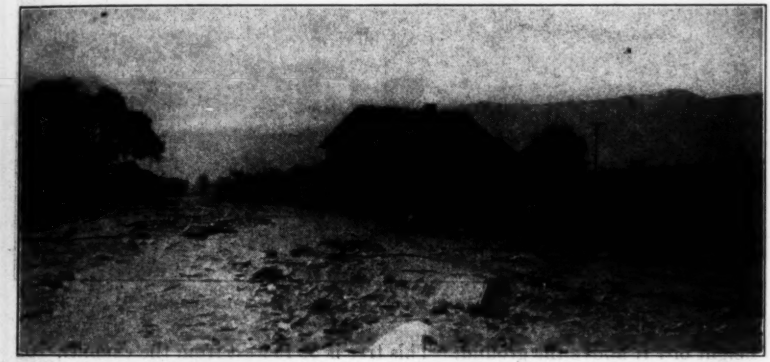

FIG. 5.—HALL-BECKLEY CHANNEL BELOW CONFLUENCE WITH SNOVER CHANNEL

Pickens Canyon.—This canyon also lies in the fire-swept Sierra Madre, about 1 mile west of Hall-Beckley Canyon. It heads on the ridge 3,300 ft east of Mount Sister Elsie, 5,081 ft high, at whose crest an automatic rain gage was placed. Flood heights were measured in this stream at six places, four of which were in the deep channel downstream from the canyon and included the entire flow.

A short distance downstream from the lowest tributary the flow section above the tops of the check dams was 621 sq ft. The water surface was 53 ft wide. At this point the channel curves through an arc of 180° with a radius of about 150 ft. Marks of the water surface were well defined and unmistakable. The super-elevation at the section was 6.75 ft; and the slope was 0.087 (see section C-3, Table 1). Downstream 60 ft the water had overtopped heavily, and eroded to a depth of 4 ft a bank of semi-consolidated gravel which is 22 ft higher than the tops of check dams. At this point the channel was 45 ft wide 20 ft below the top of the eroded bank. The catchment area at this point is 1.78 mile. Three-fourths of a mile downstream, at Mountain Street, the water was barely confined in the channel. The flow section above check dam tops (above the highest part of the check dams) was 590 sq ft (section C-5, Table 1); 100 ft downstream the water had overtopped the banks so heavily that a boulder 8 ft long, and a part of a check dam 4 ft by 4 ft by 18 ft long had been thrown entirely out of the channel, and within 20 ft of an unoccupied residence.

For one-half mile downstream from Foothill Boulevard the channel had a cross section about as shown in Fig. 6. This section of channel was much too small to carry the flow. The water overflowed so deeply at Evelyn Street that it completely scoured the bark from the upstream side of a Eucalyptus tree to a height of 7 ft above the ground. This tree is 50 ft west from the channel. On the east side of the channel, downstream, many boulders from 8 to 10 ft long were thrown out of the stream bed. The catchment area at this point is 1.82 miles. Measurements made by the Los Angeles County Flood Control District were of similar magnitude. Five sections in a distance of 140 ft gave an average flow section of 600 sq ft.[4]

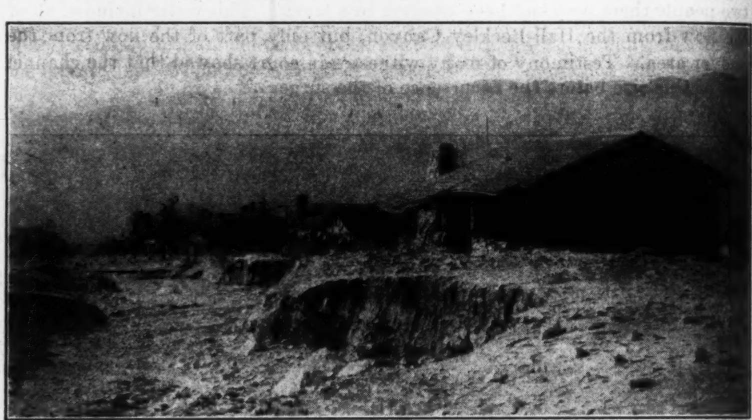

FIG 6.—VIEW OF RESIDENCE CLOSE TO PICKENS CHANNEL AFTER THE STORM OF JANUARY 1, 1934

Other Canyons.—The neighboring canyons in the Sierra Madre emitted similar surges, some of them of greater intensity than the foregoing. Drainage areas, flow sections, and hydraulic properties are shown in Table 1.

VERDUGO MOUNTAINS

The surges from the Verdugo Mountains, parallel to and about 4 miles south of the Sierra Madre, were smaller but were still excessive. They showed the response of the runoff to a variety of conditions.

A part of the Verdugo Mountains had been burned over in 1927 and vegetation had only partly recovered. The fire had denuded canyons within the zone of excess runoff as well as canyons within the normal zone. The mountain fires do not ordinarily destroy the larger trees, or the roots of chaparral, and the cover reestablishes itself faster than would be expected. The natural covering is a chapparral composed of various species of native brush, from 6 to 10 ft high. This covering breaks the direct force of a heavy rain and keeps the ground beneath covered with leaves and mulch, with a tendency both to decrease and to retard runoff.

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[4] "Los Angeles County Flood Control District Report New Year's Foothill Debris Flood, 1934," by E. C. Eaton, M. Am. Soc. C. E., report dated March, 1934, Chart No. 4 (not published).

Brand Canyon was one of the burned regions in the zone of excess runoff. It has an area of 1.06 sq miles, and lies on the south slope of the Verdugo Mountains. In this canyon a rock 87 in. long was thrown entirely out of the natural stream bed, and was lifted into a three-pronged tree near the channel. It rested there 4 ft above the stream bed. At the mouth of the canyon the flow section was 233 sq ft. The canyons adjoining Brand Canyon on the east were unburned, and vegetation was normal. Hillcrest Canyon is contiguous to Brand Canyon. Its drainage area is 0.37 mile. It discharges into a concrete street 40 ft wide. It was one of the sections best adapted to accurate measurement. No doubt existed as to the flow section, and the value of *n* could be approximated closely. The section was 84 sq ft, or at the rate of 227 ft for each square mile of catchment area (section R, Table 1). This section would have carried 2,500 cu ft per sec of clear water. Deer Canyon, also unburned, and contiguous to Brand Canyon, gave a flow section of 90 sq ft from a drainage area of 0.41 sq mile, or at the rate of 219 sq ft per mile of catchment area (section R, Table 1). The flow sections from other unburned canyons in the zone of excess runoff are shown in Table 1(*c*).

The western part of the Verdugo Mountains was not in the zone of excess runoff. Sunset Canyon is west of Brand Canyon. The two canyons were burned over at the same time. Debris carried down by the flood was impounded by a small dam in Sunset Canyon. This dam caught about 200 yd of debris from an area of 0.44 sq mile. This was at the rate of approximately 500 cu yd per sq mile. Below Brand Canyon, which was in the zone of excess runoff, 75,000 cu yd of debris per sq mile of catchment area were removed.

Rainfall on January 1, 1934.—The rainfall which caused these surges was a short violent storm of highly uncommon characteristics. At Los Angeles, about 12 miles south of the destructive area, the automatic gage of the U. S. Weather Bureau caught 7.36 in. in the wettest 24 hr. Until March, 1938, this 62-yr record showed no other period as short as 3.5 days with an equal rainfall.

Near the mountains, the gages show a surprising uniformity of rainfall over a large area, almost regardless of great differences in elevation, topography, location, and exposure. In the zone of excess runoff, and on the valley floors to the south, an average for the storm of 12.93 in. fell on an area of 80 sq miles. Approximately 12 in. of this fell in the wettest 24 hr. Almost all of the sixteen gages in the zone of excess were in the rugged mountains, at an average elevation of 2,100 ft. They showed a catch differing by 1% from the catch of the sixteen gages to the south, at an average elevation of 831 ft. The aforementioned thirty-two gages varied in elevation from 526 to 5,081 ft. This area of nearly uniform rainfall included the entire zone of excess runoff. The readings of the gages are summarized as follows:

Description	Number of gages	Rainfall for storm, in inches
Zone of Excess:		
All gages	16	12.86
Gages in 1933 burn	2	11.52
Gages close to edge of burn	5	12.02
Zone of Normal Runoff:		
Belt south of zone of excess	16	12.99
Arroyo Seco catchment east of zone of excess	5	12.12

The slight influence of elevation on the rainfall in this area during the storm on New Year's day is shown by tabulating rainfalls in the order of elevation of the gage. Placed in this order they show:

Elevtion of gages, in feet	Number of gages	Rainfall for storm, in inches
Below 1,000	11	13.46
Between 1,000 and 2,000	15	12.84
Between 2,000 and 3,000	7	12.15
Between 3,000 and 4,000	3	12.07
Above 4,000	2	12.15

The automatic gages in Haines Canyon, and on Mount Sister Elsie in the zone of excess runoff, reached their capacity before the end of the storm. They did not record its later stages, although they gave a good record up to, and for some time after, the surges. They also showed the total amount of rainfall for the storm. The automatic gage at Flintridge, from 2 to 6 miles distant from the canyons which emitted surges, was the nearest automatic gage that recorded the entire storm. It caught a total of 14.02 in., of which more than 99% fell in 40 hr, and 12.86 in., or 92%, fell in the wettest 24 hr. The gage at the Flintridge fire station is about 1 mile east of the Verdugo Canyon catchment area. It is 6 miles southeast from the gage on Mount Sister Elsie, on the edge of the area at an elevation of 5,081 ft. The Haines Canyon gage is in the center of Haines Canyon, less than a mile from that on Mount Sister Elsie, but more than 1,600 ft lower. The two latter gages are in the burned area in the mountains. Mount Sister Elsie is at the heads of Haines and Dunsmuir canyons, and within 1 mile of the heads of Pickens, Shields, and Blanchard canyons. The head of Cooks Canyon is 2,000 ft distant.

There were no noteworthy short-time rainfall intensities. One gage, several miles from the zone of excess, showed rainfall of more than 0.5 in. in 15 min. The gages within and close to the burned area showed the rainfall rates given in Table 3. The automatic gages at the Polytechnic High School and at the Weather Bureau in Los Angles, directly south from the zone of excess runoff, showed similar abrupt increases in rainfall.

The automatic gage in Brand Canyon reached its capacity before the surge, but the following notes of the observer at the mouth of the canyon supply much of the missing data: "Unusual down pour at 11:50 P.M. Dec. 31. Runoff reached flood proportions at 12:05 A.M. Jan. 1, at which time a sudden rise in volume of stream caused it to leave its channel ***." These increases, falling upon steep, thoroughly soaked, freshly burned canyons, produced the aforementioned surges. Table 3 shows the moderate rates of rainfall sufficient to cause such violent surges when other conditions are favorable for their creation. All of the surges followed promptly after the increase of rainfall. This was true not only in this storm, but also in all of the other storms noted herein. Rainfall increases during the other Southern California storms listed herein were similar. Some of the intensities are shown in Fig. 7.

TABLE 3.—STUDY OF SHORT-TIME RAINFALL INTENSITIES

Locality	Elevation, in feet	Time of increase	Rate immediately before increase	Rate immediately after decrease	Rate of increase
Haines Canyon	3,470	11:37 p.m.	0.60	1.56	0.96
Mount Sister Elsie	5,081	11:45 p.m.	0.60	1.56	0.96
Flintridge	1,325	12:01 a.m.	0.24	1.08	0.84
Average	....	....	0.48	1.40	0.92

OTHER SOUTHERN CALIFORNIA SURGES

Several less important floods of the same type have occurred in Southern California since automatic rain gages have been installed in the mountains. The first of these came on November 14, 1928, from Allen, Brand, and some small adjacent canyons burned over in December, 1927. Practically all of this area except Sunset Canyon was in the zone of excess runoff on January 1, 1934. Except in Sunset Canyon, peak intensities seem to have been not greatly different from the 1934 surges, but the entire duration of the flow was not greater than 20 min.

In Allen Canyon four measurements by the Flood Control District gave an average flow section from 0.2 mile of 95.25 sq ft. This was at the rate of 476 sq ft of section per square mile of catchment area. In Brand Canyon the surge was stated to have been higher than that of 1934.

The rainfall which caused these surges was measured by sixteen gages within 5 miles of the zone of excess runoff. None showed a total catch greater than 1.45 in. The two automatic gages in the vicinity showed a maximum 10-min rate of 1.62 in. per hr, although the maximum 5-min rate was 2.52 in. per hr. In February, 1929, Brand Canyon emitted a much smaller surge during a rain which would almost pass unnoticed under ordinary conditions. The automatic gage then in the canyon showed a rainfall starting suddenly, with a maximum rate of 1.80 in. per hr. The entire rainfall lasted only 15 min, and the total rain on the 1.07 sq mile was less than 12 acre-ft.

In October, and again in December of 1934, the then recently burned area in the Sierra Madre yielded surges much smaller than those of January 1. The automatic gage in Haines Canyon showed a maximum rainfall rate of 1.68 in. per hr in December. In October this gage showed a 5-min rain at the rate of 3.5 in. per hr.

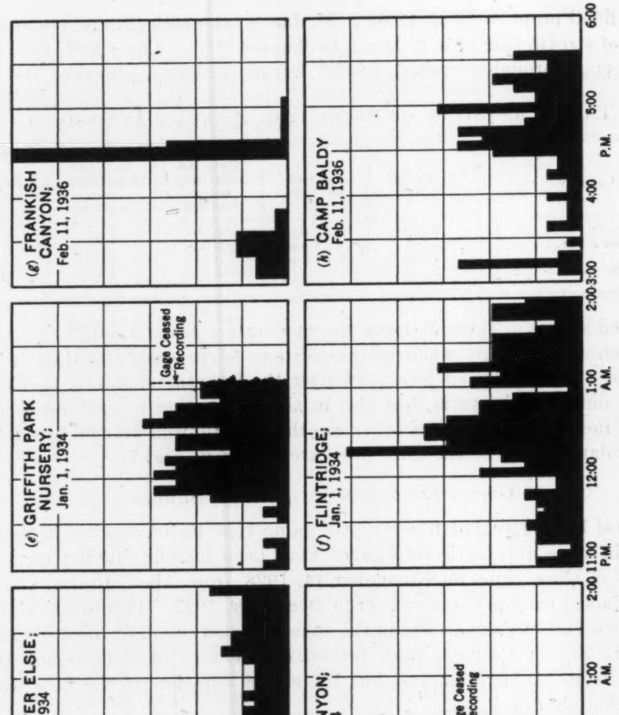

FIG. 7.—RATE OF RAINFALL FOR EACH 5-MIN PERIOD DURING THE TIME CONSIDERED

A smaller flood of this character occurred about 30 miles to the east on the south slope of the Sierra Madre on February 11, 1936. This flow came from a then recently burned canyon (burned in 1935) with an area of 0.56 sq mile. The surge was estimated by witnesses to be 10 ft high. None of the neighboring unburned canyons yielded unusual runoffs.

This flow was caused by a sudden rain beginning on recently burned, wet canyon slopes. For 7 min rain fell at a rate of 3.00 in. per hr. It then dropped for 5 min to 1.32 in. per hr, and then to a light sprinkle. During the first 7 min less than 10 acre-ft of rain fell on the entire canyon. The total volume of rain for the 12 min was less than 13 acre-ft, which fell at a maximum rate of 1,000 cu ft of water per sec.

Other Floods.—Engineering publications contain accounts of many floods in the United States in which, apparently, the runoff peak cannot be expressed as a percentage of the rainfall on the area. In these descriptions are included more than forty floods in which peak flows have been measured at quantities ranging from 1,000 to 4,170 cu ft per sec for each square mile of catchment area. More than one half of these runoff peaks have been reported from east of the Mississippi River. In some of these cases, the absence of automatic rain gages makes it uncertain whether the runoff peak was caused by an accumulated surge, or as the direct result of an excessive rate of rainfall for a period longer than the concentration time of the area. It is difficult or impossible to reconcile many of these runoff figures with the rainfall measurements. Some of the accounts seem to show definitely that the conditions necessary to an accumulated peak were present. It is possible that a reexamination of the data would show these conditions in other cases.

Only three of the floods which have been reported, and which probably are of this character, are listed herein (Cardens Bluff, Tenn., and Northern Utah in 1923 and in 1930), although many of the other floods reported seem equally to belong in this classification.

Cardens Bluff.—The flood on the Watauga River, in eastern Tennessee on June 13, 1924, is the first.[5] At this place 14.98 in. of rain fell in about 8 hr. About four fifths of this rain fell between 6:30 p.m. and 10:00 p.m. The rain then decreased, but did not stop entirely until 12:30 a.m., when a second hard rain began that lasted for 2 hr. A small ravine, whose catchment area is about 0.025 sq mile, emitted a surge 8 to 10 ft in height which totally demolished two houses and killed nine of the occupants.

Northern Utah.—Several floods which originated in the mountains north of Salt Lake City, Utah, in 1923 and in 1930 have been the subject of investigation and report.[6, 7, 8, 9, 10] The first of these floods occurred on August 13, 1923, in Farmington, Willard, and other canyons along the steep western front of the Wasatch Mountains facing Great Salt Lake. Late in the afternoon of that day an exceedingly abrupt, heavy rain brought enormous, short-time flows from several of the canyons, destroying nine lives and much property. At Farmington the flood arrived within 20 min of the beginning of the rain at that place, doing great damage. At Willard Canyon, the discharge was even greater and more destructive. The canyons are steep, and largely denuded by overgrazing.

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[5] "Record Cloudburst Flood in Carter County, Tenn., June 13, 1924," by Warren R. King, *Monthly Weather Review*, June, 1924, p. 311; also "Surface Waters of Tennessee," by Warren R. King, State of Tennessee, Division of Geology, *Bulletin No. 40*, 1931, p. 61.

[6] "The Floods of 1923 in Northern Utah," by J. H. Paul and F. S. Baker, *Bulletin* of the Univ. of Utah, Vol. XV, No. 3, March, 1923.

[7] "Floods at Farmington and Willard, Utah," by J. Cecil Alter, *Monthly Weather Review*, Vol. 51, August, 1923, p. 420.

[8] "Torrential Floods in Northern Utah," by a special commission, S. Q. Cannon, M. Am. Soc. C. E., Chairman, Agricultural Experiment Station, Utah State Agricultural Coll., Logan, Utah, *Circular No. 92*, January, 1931.

[9] "Floods and Accelerated Erosion in Northern Utah," by Reed W. Bailey, C. L. Forsling, and R. J. Becraft, U. S. Dept. of Agriculture, *Miscellaneous Publication No. 196*, August, 1934.

[10] "Mud Floods in Utah," by J. Cecil Alter, *Monthly Weather Review*, Vol. 58, August, 1930, p. 319.

The storm that caused these floods was a short, sharp shower, which yielded less than 1.5 in. of rain. At Salt Lake City, 16 miles south, the rainfall was the most intense on record. Most of the rain fell in 15 min at an average rate of 3.04 in. per hr.

Floods of 1930 in Utah.—In July and August, 1930, the same district was again swept by short-lived flows which swept boulders weighing 200 and 300 tons, together with smaller rocks, out of the canyons. These floods were even more severe than that of 1923. The fronts of the waves were reported to be several feet high and were not preceded by a flow of water. The storm of August 11, 1930, was also a short downpour. Ogden received 1.72 in. of rain, 0.40 in. falling in 9 min. This was at the rate of 2.67 in. per hr.

RECAPITULATION

The various Southern California surges permit of quantitative comparison because of the similarity of some of their conditions. In each of the cases listed, the flood has occurred in a steep canyon, after an abrupt increase of rainfall. For the purposes of comparison, the severity of the surges has been placed in four classifications (see Table 4). In classifying these surges, account has been taken, not only of the flow sections, but also of the properties of the channels, and of the comparative destructiveness of the flows after they left the canyon mouths.

TABLE 4.—COMPARATIVE SEVERITY OF SURGES

Locality	Date	Time	Condition	Age of burn, in months	Slope drop in first mile, in feet	Severity
Brand-Allen-Sunset	11/14/28	9:25 a.m.	Burned	11	1,700	Heavy
Brand	4/ 4/29	9:40 a.m.	2/3 burned	16	1,700	Light
Sierra Madre	1/ 1/34	12:01 a.m.	Burned	1.5	1,700	Violent
Brand	1/ 1/34	12:01 a.m.	2/3 burned	72	1,700	Heavy
Hillcrest-Deer	1/ 1/34	12:01 a.m.	Not burned	....	1,700	Moderate
Baldridge-Whiting	1/ 1/34	12:01 a.m.	Not burned	....	1,700	Light
Sierra Madre	10/17/34	4:00 p.m.	Burned	11	1,700	Light
Sierra Madre	10/18/34	2:00 a.m.	Burned	11	1,700	Moderate
Fern	10/18/34	2:00 a.m.	Burned	3	1,700	Heavy
Sierra Madre	12/14/34	7:00 a.m.	Burned	13	1,700	Light
Frankish	2/11/36	4:45 p.m.	Burned	6	1,700	Heavy
Sycamore	1/ 1/34	12:01 a.m.	Not burned	....	865	None

CONCLUSIONS

In searching the evidence disclosed by these surges, many factors emerge as having a strong bearing on their occurrence and size. Some of the factors involved can be evaluated roughly, but several of them can only be recognized as important, though unmeasured, items in the final result.

The combination of conditions necessary to the formation of the surges is not common. They are not especially a problem of the climatic and topographic environment of the west. Whenever rain increases abruptly to a sufficient intensity upon steep slopes, the formation of the surges may be expected. Small, denuded areas facilitate their creation. When all of these conditions are present, a torrential rainfall is not required. If any of the conditions are missing, or are present to a diminished degree, the surges are reduced in size or are suppressed entirely. On the other hand, a torrential rainfall causes them even though other conditions are not favorable. When conditions are favorable, one of the most striking features of these floods is the disparity between the magnitude of the surges and the rate of rainfall at the time of their occurrence.

When the effects of the storm of January 1, 1934, were studied both in the Sierra Madre and in the Verdugo Mountains, four facts stood out with special prominence:

(1) The unexpectedly large flow sections;

(2) The lack of conformity between the various burns and the zone of excess;

(3) The nearly simultaneous formation of the surges in the various canyons; and

(4) The countless evidences of high velocity.

The first of these has already been discussed. The channels shown in Figs. 4 and 5 apparently were not half large enough to carry the flow.

LACK OF CONFORMITY WITH "BURNS"

Lack of conformity between the area burned in 1933 and the zone of excess runoff was not noted in the Sierra Madre Mountains; but in the Verdugo Mountains the area burned in 1927 apparently did not coincide at any point with the zone of excess runoff.

The surges occurred in all of the canyons investigated in a total area of more than 10 sq miles. They did not form in some burned canyons outside of this area, but did form in other nonburned canyons within the area. Brand Canyon and Sunset Canyon were both in the 1927 "burn." Sunset Canyon was not in the zone of excess runoff in the storm of January 1, 1934.

The lack of conformity also extended to nonburned areas. Attention has already been called to the surges from the nonburned part of the Verdugo Mountains. Within the zone of excess, these canyons discharged quantities of debris only slightly less than from the burned areas. Below Hillcrest Canyon, and the smaller canyon to the east, both not burned, the debris removed was at the rate of more than 60,000 cu yd for each square mile of catchment area (see section R, Table 1). The evidence on the ground seemed to show clearly that the "burn" was an exceedingly important item, but that it was not the only, or even an essential, factor in the formation of the surges.

NEARLY SIMULTANEOUS FORMATION OF THE SURGES

The "nearly simultaneous formation of the surges" is interpreted herein as indicating definitely that the cause of the surges was the change in rainfall conditions. No other factor seems to account for their formation at this particular time, in all of the canyons within this comparatively wide area, without regard to differences in the size of the various canyons, in elevation, or in the condition of the vegetation.

The surges occurred in all of the cases investigated soon after the abrupt increase of rainfall, and not at the culmination. In this respect, as in many others, they differ from the usual type of flood, which comes to a peak at a much later stage of the rainfall.

VELOCITY

There were many indications of the surface velocity. Splash marks from 10 to 14 ft above high-water level were noted and photographed at almost all points along the natural channels. Near the lower end of the Pickens Channel where the erosion and destructiveness had decreased, the flow had piled rocks and sand on to a roof which stood nearly in the path of highest velocity. The rocks were lodged 10 ft above the high-water mark, and had been thrown forward at least 11 ft. Below Cooks Canyon a boulder had been thrown into a tree 8 ft above high-water marks. Its horizontal movement was not less than 10 ft.

This material left the water surface at velocities of 25 to 30 ft per sec. Apparently, surface velocities were not much lower than would be indicated by the Manning formula. Velocities in the lower part of the stream are conjectural. The flows were carrying very heavy percentages of debris. The extent to which the bed load slowed this part of the flow is not known.

CONDITION OF SURFACE

In a canyon denuded of vegetation, the response of the stream to changing rainfall conditions is greatly accelerated, and increased rainfall is translated into increased runoff much more quickly than is the case where the ground is covered with vegetation.

Although a denuded surface invites the formation of these surges, they form, under conditions favoring them, on semi-denuded, or even on completely covered areas.

Hillcrest, Deer, Baldridge, and Whiting canyons were fully covered on January 1, 1934, with the normal chaparral growth. The brush cover increases in density from Hillcrest to Whiting Canyon, and the surges from these canyons showed a steady decrease from Hillcrest to Whiting Canyon.

The floods of 1924, in eastern Tennessee, occurred in a densely wooded environment, but rainfall rates were so high that the cover did not give sufficient protection, and very violent surges occurred.

Brand Canyon was burned in 1927. At the time of the flood on New Year's day, 1934, this vegetation had recovered during a 6-yr period. Many photographs of this canyon taken during the investigation following the New Year's storm seem to indicate that the brush had recovered about one half of its normal protective effect. During that storm, the surge from Brand Canyon was much smaller than from the then recently burned canyons.

STEEPNESS OF CANYONS

The important part played by the steep slope is illustrated by Sycamore Canyon, 1 mile east of the nonburned canyons in the Verdugo Mountains. Gradients in Sycamore Canyon are much lower than in the zone of excess runoff. This is true, not only of the stream bed, but of the side slopes as well.

So far as noted, the lower gradient was the only respect in which Sycamore Canyon was less favorable to the formation of the surges than the four nonburned canyons in the Verdugo Mountains. Three rain gages showed the rainfall of the storm of January 1, 1934, to be somewhat greater than in the canyons which emitted surges. One of these was an automatic gage and showed that rainfall characteristics were similar. The brush covering was complete, but not so dense as in the nonburned canyons in the Verdugo Mountains.

The entire runoff from this canyon flowed down a concrete street. The runoff peak, per square mile of catchment area, was not more than one tenth as great as the peak from the nonburned Hillcrest Canyon, also measured in a concrete street.

DURATION OF PEAKS

Various evidence upon the ground, as well as statements of many witnesses, showed that maximum peaks in the flood of January 1, 1934, were of short duration.

In Haines Canyon, a reliable approximation can be made of the total material in the wave front. At the mouth of the canyon, about one fourth of a mile downstream from the concrete gaging station, an old gravel pit had been utilized by the Flood Control District so as to make a suitable debris basin. It had an area at the spillway level of 2.13 acres. Throughout the evening of December 31, the debris basin was observed by a watchman of the adjacent rock crushing plant, and also by an employee of the Flood Control District who was detailed to watch the water level in the basin. Just before the surge water level was from 4 to 5 ft below the spillway. The surge raised the level from that point until it flowed 2 ft deep over the spillway. The indications are that the surge in Haines Canyon contained from 15 to 17 acre-ft before the flow dropped to a point where it was carried by the spillway without raising the level in the basin. This value includes water and debris, but not entrained air (section J, Table 1).

DEBRIS CONTENT

The surges were very heavily charged with debris. Most of the witnesses who placed a value on the debris content estimated the flow as 50% water and 50% solids. Three observers noted the character of the flow on the streets below Hall-Beckley Canyon. One of them estimated it as 50% solids, whereas another who had seen the same flow at the same time estimated it at 60% solids. E. C. Eaton, M. Am. Soc. C. E., who saw the flow near this point, estimated the solids as from 50% to 70% of the total.[11] The flow from Shields Canyon was estimated by one observer to be 50% solids, and to be "definitely thicker than water."

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[11] "Los Angeles County Flood Control District Report New Year's Foothills Debris Flood, 1934," by E. C. Eaton, report dated March, 1934, p. 17-C (not published).

In Deer Canyon an observer stated that the flow "wasn't like water *** like a thin mixture of cement with rocks." Another observer said there were "plenty of rocks on the bottom but the flow itself was distinctly 'water.'" In Pickens Canyon the flow was stated to be "very muddy but splashing."

ACKNOWLEDGMENT

Indebtedness in the collection of data, or in the preparation of this paper, is acknowledged to Edward M. Lynch, to Charles G. Frisbie, and to Allen Van Rensselaer. Mr. Frisbie joined with the writer, especially, in making the measurements recorded in Table 1. Much valuable assistance has also been given by the Los Angeles County Flood Control District, the California Forest and Range Experiment Station of the U. S. Department of Agriculture, the U. S. Weather Bureau, the U. S. Geological Survey, as well as by many others.

DISCUSSION

Ivan E. Houk,[12] **M. Am. Soc. C. E.** (by letter).—The author has contributed a novel treatment of an old subject. Cloudburst floods and the almost instantaneous surges which often accompany them have been described in American scientific literature for more than a century. However, so far as the writer knows, the author's comparison of such phenomena on the basis of the maximum cross-sectional area per square mile of catchment basin is new. Such a basis of comparison is logical and about as practicable as any that might be adopted, especially when the intense rates of rainfall and runoff occur near the head of a canyon and can reasonably be assumed to extend over the entire catchment area. When this is not the case, as often happens, it would seem better to base the comparison on the maximum cross-sectional area per square mile of zone of intense rainfall and resulting intense runoff.

It is always desirable, when at all feasible, to estimate the maximum rate of runoff from the contributing territory; but, unfortunately, flow estimates can seldom be made with satisfactory degrees of accuracy in cases of cloudburst surges like those described by the author. Uncertainties regarding roughness factors, surface slopes, acceleration slopes, velocity distribution, and effects of transported debris are so great that flow calculations are extremely questionable, even when the maximum cross-sectional areas can be determined accurately. For this reason, the publication of data on cross-sectional areas, together with contributing catchment areas, for cloudburst flood waves of unusual magnitude, is valuable to the hydraulic engineer. The reader, if he wishes, can make his own estimate of the average velocity at the location of measurement, and can then calculate the probable rate of discharge and corresponding probable rate of runoff from the contributing drainage area above the cross section.

Cloudburst surges are not confined to mountainous regions. They occur in practically all sections of the United States. The writer has personally witnessed such phenomena in the gently rolling topography of Iowa and Ohio, as well as in the comparatively steep topography along the east edge of the Colorado Rockies. However, in mountainous territory, destructive flood waves are caused by lower rainfall intensities, due to the more favorable conditions for rapid runoff. The rates of precipitation which caused the flood waves described by the author would seldom, if ever, cause destructive flood waves in the comparatively flat topography of the central Mississippi River Valley. Furthermore, cloudburst surges in mountainous regions transport much heavier loads of sand, gravel, and boulders than surges of comparable magnitude in regions of gently rolling topography where rates of movement are necessarily somewhat lower. Cloudburst surges in any locality may be intensified greatly if the storm moves downstream at about the speed of the flood wave.

One of the first cloudbursts recorded in the United States was the storm that occurred at Catskill, N. Y., during the evening of July 26, 1819, when approximately 13 in. of rain fell in three hours. Although the most intense precipitation fell relatively close to the Hudson River, the flood waves from small tributary drainage areas were great enough to erode new channels at several locations.

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Another storm of historic interest was the one that occurred in Delaware County, Pennsylvania, about 20 miles southwest of Philadelphia, during the afternoon of August 5, 1843. Cloudbursts that afternoon precipitated a total rainfall of approximately 16 in. in three hours and caused the highest local floods known in Delaware County since the year 1700 A.D. Flood waves, several feet high, with almost vertical fronts, passed down some of the small creek channels which drain that section of Pennsylvania.

The storm which centered in Des Moines County, Iowa, about 15 miles northwest of Burlington, between 2 and 4 a.m. on August 16, 1898, was similar to the aforementioned Pennsylvania storm. Field investigations by Maurice Ricker,[13] reported to the Iowa Academy of Science, showed that approximately 16 in. of rain fell in less than three hours over an area of about 50 sq miles. The resulting flood runoff destroyed trees 12 in. in diameter, moved large rocks many feet from their original locations, cut new stream channels in some places, and caused damages estimated at $100,000.

Since the beginning of the twentieth century, many cloudburst storms and resultant intense floods have been described in engineering literature. Such descriptions have come from practically all parts of the country. A few typical examples which may be cited include the Heppner flood of June 14, 1903, in Oregon; the Salt Creek flood of September 28, 1923, in Wyoming; the Devil's Creek flood of June 10, 1905, in Iowa; and the Cabin Creek flood of August 9, 1916, in West Virginia. More recent examples include the flood of June 3, 1935, at Mexico City, Mexico; the Bear Creek Canyon floods of July 8, 1933, and September 2, 1938, near Denver, Colo.; the Tehachapi Creek flood of September 30, 1932, in Kern County, California; the Republican River flood of May 31, 1935, in Nebraska; and the floods of July 7 and July 8, 1935, in south central New York. Many others might be mentioned. Undoubtedly, many isolated storms of cloudburst nature have occurred in sparsely settled communities and have never been reported because the resulting flood flows did not greatly damage human life and property.

Since cloudburst surges are caused by cloudburst rainfall, the rates of flow and total runoff during such occurrences can always be reconciled with the rainfall, providing all pertinent facts are known. This is axiomatic. Difficulties in reconciling cloudburst runoff with cloudburst rainfall are due to uncertainties in data. During such phenomena all factors involved are so variable that sufficient accurate information can seldom be secured. Rates of rainfall, areas on which such rates of rainfall occur, rates of runoff, areas contributing such rates of runoff, and respective time data for both rainfall and runoff are all uncertain. In attempting to correlate runoff with rainfall during such storms, the actual area on which the intense precipitation takes place should be considered, rather than the total drainage area above the location of stream measurement.

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[13] "The August Cloud-Burst in Des Moines County," by Maurice Ricker, *Proceedings*, Iowa Academy of Science, Vol. 6, 1898.

During intense storms of cloudburst nature, one automatic rain gage does not tell the entire precipitation story, even in relatively small catchment basins of one square mile or less. During such storms, rain gages, only 100 ft apart, may show differences in precipitation of more than 100%. It is not only possible, but in the writer's opinion also highly probable, that the rates of rainfall which caused the surges described by the author were considerably higher in some parts of the catchment basins than those actually measured at the gages.

Careful studies of intense storms show that abrupt increases in rainfall and runoff occur in some part or parts of the storm area in practically all storms of cloudburst nature, although such conditions may not be accurately measured and scientifically reported. The writer believes that the combination of conditions necessary to the formation of surges is a common characteristic of such phenomena. Consequently, he does not consider it logical to classify the floods of southern California, northern Utah, and Cardens Bluff as a special type of cloudburst phenomena.

The engineering profession is indebted to the author for his clear presentation of valuable rainfall, runoff, and erosion data pertaining to the southern California floods of January 1, 1934.

GORDON R. WILLIAMS,[14] ASSOC. M. AM. SOC. C. E. (by letter).—The phenomenon with which this paper deals is a special type of flood wave which, in the regions under consideration, is also in part a debris wave. A serious deficiency in the paper is that the author has failed to separate the characteristics of the wave from those of the flow of water which follows the wave. By tabulating the hydraulic properties of the wave fronts, the implication is made that these values can be substituted in a formula for normal open-channel flow (the Manning formula) and that the results will indicate that phenomenal rates of water discharge, entirely inconsistent with the recorded rates of rainfall, have occurred. The purpose of this discussion is to reveal, in the light of present knowledge of flood waves, the inadvisability of such a procedure.

A flood wave with a turbulent, vertical front has been designated by different terms by various writers on the subject. Harold A. Thomas,[15] M. Am. Soc. C. E., uses the term "hydraulic bore"; Hunter Rouse,[16] Assoc. M. Am. Soc. C. E., the term "gravity shock wave"; and A. Schoklitsch[17] the term "flood surge." The term bore will be used in this discussion.

For a bore to be formed it is necessary that there be a sudden concentration of water in a channel and that conditions in the channel be such that the concentration of water will produce a depth which is more than twice the initial depth of flow. This condition is shown diagrammatically in Fig. 8, in which

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[14] Associate Hydr. Engr., U. S. Engr. Office, Baltimore, Md.

[15] "The Hydraulics of Flood Movements in Rivers," by Harold A. Thomas, *Engineering Bulletin*, Carnegie Inst. of Tech., 1934, pp. 24, 28–29.

[16] "Fluid Mechanics for Hydraulic Engineers," by Hunter Rouse, *Engineering Societies Monographs*, McGraw-Hill Book Co., Inc., New York, N. Y., 1938, pp. 382–388.

[17] "Hydraulic Structures," by Armin Schoklitsch, published by A.S.M.E., 1937, pp. 121–128.

d_2, the maximum depth of the wave front, is more than twice d_1, the initial depth of flow. If $d_2 < 2\,d_1$, the wave front assumes a normal wave form, and no bore is formed.

The author expresses the theory that a bore is formed when there is an abrupt increase in rainfall rates following a period of prolonged rainfall. This theory can be substantiated in part because, if there has been rain for some time, high initial infiltration rates have been succeeded by lower rates, much depression storage has been taken up, and conditions are "ripe" for quick concentration in the stream channels of the increased rainfall. If a stream flows through a narrow canyon, where there is little valley storage, the concentration of water will produce a depth which is large in relation to the depth of the initial flow, and a bore will be formed. Thus it is evident that there must be both a meteorological and a hydraulic cause for a bore and that the increased rainfall is only the meteorological cause.

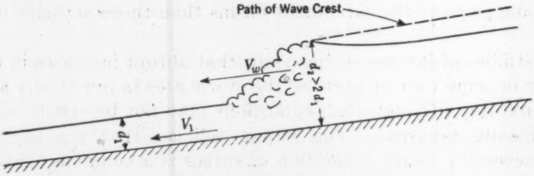

FIG. 8.—PROFILE OF A DEBRIS-FREE BORE

Conditions in the burned areas mentioned in the paper were probably more conducive to rapid concentration of water in the upstream channels than in the unburned areas, but, as will be indicated subsequently herein, such conditions may have had a more marked effect on the form of the flood wave and on the debris movement than on peak rate of water discharge.

The discussion thus far has considered a bore which is composed entirely of water. The bores that occur in the western mountains pick up tremendous quantities of debris and carry them along in suspension or as bed load. Where there are steep bottom slopes it is not impossible that debris may occupy 50% or more of the cross-sectional area of the wave front.

Where the debris content is extraordinarily large, the solids may act as a moving dam, the effect of which would account for some of the large cross-sectional area below the floodmarks. Any debris left behind by the initial wave would be carried on by the less heavily burdened water which followed the wave, and thus remove the last evidence of the true water area at the maximum stage. Evidently the movement of debris was greater from the burned areas than from the unburned areas. In extreme cases the conditions were probably not unlike the "detritus avalanches" of mountainous European streams discussed by Otto Franzius.[18]

Statements in regard to the consistency of the fluid mass are contained in the last sections of the paper but should have been given more prominence in the first part, especially when the channel areas below the floodmarks are considered. Descriptions of the debris waves in La Cañada Valley, California, flood and theoretical discussions of their characteristics have been presented by Harold C. Troxell, Assoc. M. Am. Soc. C. E., and John Q. Peterson.[19]

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[18] "Waterway Engineering," by Otto Franzius (translated by Lorenz G. Straub, Assoc. M. Am. Soc. C. E.), *The Technology Press*, Mass Inst. of Tech., Cambridge, Mass., 1936, p. 15.

For the purpose of further discussion, let it be assumed that the cross-sectional area of the wave front is composed entirely of water. Under conditions which exist in mountain canyons it is entirely probable that the depth of the bore is greater than the depth of the less turbulent water which is flowing after it, as indicated in Fig. 8. Some of this greater depth is no doubt due to entrained air. The profile defined by the floodmarks would be as shown in the dotted line in Fig. 8. If the slope of such a profile is used in the Manning formula to determine the mean velocity of the water particles following the wave, and if the area under the profile is used as the area of flow for these water particles, an obviously incorrect result will be obtained. The velocity of the wave front is a velocity of wave propagation, and, except in the case of a dry channel, is always greater than the velocity of the water particles that follow. The velocity of wave propagation cannot be obtained by the Manning formula and, if it could be so obtained, it could not be applied to the cross-sectional area of the wave to determine the discharge in the channel back of the wave. Mr. Schoklitsch has derived a formula for the velocity of the bore, V_w, which may be written in the form

$$V_w = V_1 + \sqrt{g\left[d_1 + \tfrac{3}{2}\,(d_2 - d_1)\right]} \ldots\ldots\ldots\ldots\ldots (2)$$

The water in a bore is in a state of extreme turbulence, largely in the form of a roller with a horizontal axis. Water is supplied to the bore both from the water that is being overtaken and from the following flow.

For the following reasons, the channel areas, wetted perimeters, hydraulic radii, and surface slopes given in this paper cannot be used to determine peak discharges when the form of the flood wave is a bore:

(*a*) The area during the passage of the wave front probably contained a large percentage of solids in transit and in some cases there may have been a damming effect;

(*b*) The water in the maximum area was in a state of extreme turbulence, characteristic of a vertical wave front, and the average velocity of the water particles relative to the stream bed was not that of the wave front;

(*c*) The area was larger than the area occupied by the less turbulent water which followed the wave; and

(*d*) The surface slope obtained from the floodmarks was defined by the crest of the wave and may not have been indicative of the slope of the water back of the wave.

The writer realizes that this discussion offers no solution for obtaining the discharge following a bore. The nearest approach would seem to be in measuring short time changes in storage in debris basins and reservoirs at the time of a flood. The rate of inflow of water could be obtained by subtracting from the increase in storage the increase in the quantity of solids that were stored. The difficulty would lie in the fact that it would be necessary either to assume that all the increase in the volume of solids came at the time the peak entered the storage basin or to assume that the increase in solids was proportional to the changes in total storage; and neither assumption would be strictly correct.

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[19] "Flood in La Cañada Valley, California, January 1, 1934," by Harold C. Troxell and John Q. Peterson, U. S. Geological Survey *Water Supply Paper No. 796–C*, 1937.

There are several places in the paper where the use of terms or descriptive phrases is not in accord with accepted usage in the science of hydrology. In Fig. 1 the depth of flow preceding the "surge" is designated as "depth of runoff." This latter term is always applied to the volume of water which an area yields in a storm, or longer period of time, expressed as a uniform depth in inches over the entire area. Use of the term "depth of runoff" in Fig. 1 is misleading.

Again, under the heading "Flood of January, 1934," there is reference to soil having attained a "high degree of saturation." Except in a special chemical sense, it is not possible to have a "degree of saturation." Furthermore, the use of the word "saturated" is not advisable in reference to soil cover as experiments have indicated that infiltration continues at a reduced and steady rate even after a period of prolonged rainfall. However, it is conceded that, if the soil forms only a thin layer over rock, infiltration may become negligible.

DONALD M. BAKER,[20] M. AM. SOC. C. E. (by letter).—This paper adds some interesting and valuable information to the small amount now available in connection with the phenomenon designated by Nathan C. Grover, M. Am. Soc. C. E.,[21] as the "standing flood wave," and points to the following conclusions:

1. Protection against flood flows may be entirely inadequate under certain combinations of physical and meteorological conditions. This applies particularly to floods from small, steep, drainage areas, which depend solely upon improved channels, the capacities of which have been developed by the usual rational method (which involves such factors as intensity of precipitation, time of concentration of flow, percentage of rainfall appearing as runoff, etc.).
2. The waves or surges described in the paper, although of great magnitude and destructive force, are of relatively small volume or cubical content.
3. Denudation of the surface of the drainage area of vegetal cover by fire or other means, although not the sole cause of the formation of such waves, apparently results in a great increase in their magnitude and destructive force.
4. In view of the magnitude of such waves, it would not appear that the construction of check dams in the main channels—even if they were of a permanent nature—would serve any useful purpose as a protective measure against their occurrence or formation; nor would such a program involve complete coverage of the entire drainage area. If the cost of such construction could be kept within reasonable limits, construction of a permanent type of check dam in the smaller channels over a portion of the drainage area might serve to increase time of concentration of the water flowing from such checked

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[20] Cons. Engr., Los Angeles, Calif.

[21] "Standing Waves in Rivers," by Nathan C. Grover, *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1400.

area, and so reduce peaks, although it is questionable if such measures would be as economical as construction of debris basins.

The combination of physical and meteorological conditions which produced the waves described in the paper, in the canyons draining the area burned in 1933, included:

(*a*) An abrupt increase in rainfall intensity over a wide area, amounting to nearly 1.0 in. per hr, or 260% of the preceding intensity rate;

(*b*) A state of almost complete saturation of the surface of the drainage area at the time of the occurrence of the increase in rainfall intensity, resulting in a runoff coefficient approaching 100%;

(*c*) An almost complete absence of brush and litter on the surface of the drainage area, except in the beds of the channels, this condition affording opportunity for high velocity of surface runoff;

(*d*) Channels located in easily eroded material, and containing a considerable amount of debris accumulated over a long period; and

(*e*) Steep surface slopes on the drainage area, ranging from 25% to 50%, or more, and high channel gradients throughout, the gradients being somewhat less in the lower reaches of the canyons.

If it is assumed that the runoff on the slopes and in the channels follows the Manning formula, it can be shown that, with conditions of constant slope and roughness, where depth of flowing water approximates the hydraulic radius, such depth varies with the 0.6 power of the quantity, and velocity with the 0.3 power of the quantity.

With a 100% runoff factor, an increased intensity equal to 260% of the preceding intensity will increase the rate of runoff in the same proportion, and under the foregoing assumptions depth would be increased 178% and velocity of flow 133%. Furthermore, since slopes and channel gradients in the higher elevations are greater than at lower levels (probably upwards of 50%), the time of concentration of water in the upper reaches of the drainage area would be shorter and runoff from increased rainfall would accumulate in the channels more quickly at higher elevations. The fan-like shape of the heads of the canyons would also tend to increase this rate of accumulation.

Were all of the channels on the drainage area straight and smooth, offering little frictional resistance, the increased flow would have a tendency to "slide" smoothly along the channels, with a gradual "welling up" of the flow. However, the channels are crooked, rough, and filled with boulders and trees. These conditions result in extremely turbulent flow, particularly in the first increments of increased flow, the energy of the water being dissipated and its velocity reduced. Furthermore, some of the first portions of the increased flow are accumulated as channel storage; and the rising stream picks up debris, particularly brush and trees, which tend to pile up and retard the flow. The general result of these conditions is that the first increments of increased flow do not move downstream as fast as does the later discharge. This flow, especially the portion at the surface and near the center of the stream, moves faster than the water on the sides and bed, and soon a small wave is created which is very turbulent in character and filled with debris. The faster moving water upstream spills over this wave, its velocity being dissipated.

The cumulative effect of these conditions is the formation of small standing waves in each of the small channels, which concentrate into larger waves and ultimately into a large standing wave in the main channel. This large standing wave acts as a barrier or moving "dam" as it advances downstream at a velocity less than that of the water behind it. This onflowing water keeps piling up higher and higher behind the "dam" thus formed by the standing wave.

Because of its turbulence, the water along the wave front has much greater erosive and transporting power than has the water flowing behind it, and is able to pick up and move large loads of debris. Large boulders such as are described in the paper are set in motion and literally "bounced" down the channels.

The fact that this advancing wave carries much material considerably larger than sand throughout its entire cross section is evidenced by the existence of small rock chips, ⅛ in. or more in diameter, firmly embedded flush with the surface of a hard-pine telephone pole to heights of 15 ft or more above the ground, the pole being located on the north side of Foothill Boulevard just west of the Pickens Canyon crossing.

Three considerations lead to the conclusion that the volume of water in these standing flood waves was not great, and that their duration was very short. The cross section of the flooded channel in Haines Canyon at the U. S. Geological Survey gaging station, as given by the author, had an area of 299 sq ft, with a depth of water between 11 and 12 ft. The capacity of the Haines Canyon debris basin between the elevation where the water stood prior to the occurrence of the wave and the maximum depth of overflow of about 2 ft on the spillway crest is apparently between 15 and 17 acre-ft, or from 650,000 to 740,000 cu ft. A depth of 2 ft over the spillway crest would represent a flow of the order of 600 cu ft per sec. A discharge of 5,000 cu ft per sec in excess of this quantity would have filled the basin in a little more than 2 min, and one of 10,000 cu ft in a little more than 1 min.

If Manning's formula is used, and the hydraulic properties of the channels and cross sections as given in Table 1 are taken, velocities ranging from 20 to 70 ft per sec are developed. Such velocities would have far greater erosive effect than is evidenced on the ground or in photographs taken immediately after the flood. It is probable that the slope of the water surface upstream from the front of the wave was considerably less than the natural slope of the channel. This would cause the wave to take the shape of a wedge, with a base equal in area to the cross section of the channel, and a height equal to the distance between the wave front and the point upstream where the surface slope of the flowing water was equal to that of the natural channel.

At the location where Pickens Canyon crossed Foothill Boulevard, a concrete box culvert about 11 ft by 18 ft in cross section carried the wash under the paved highway, with about 4 to 5 ft of earth fill over the top of the culvert. Apparently water reached a height of 15 ft or more over the paved section of the highway. Even with velocities of 15 to 20 ft per sec, a flow of any duration undoubtedly would have washed out the highway embankment and caused great erosion downstream from it; but such was not the case.

The ratios of cross-section areas of channels draining lands burned in 1933 and 1927 (with partial recovery of brush cover), and unburned areas was 442, 166, and 100%, respectively. The effect of fire upon the magnitude of the flood waves, due to denuding the surface of the drainage area, is thus indicated forcibly. Although a stand of brush recovers fairly rapidly in Southern California, it requires a much longer time to accumulate litter, which has great retarding effect upon runoff.

With depths of cross sections in the channels draining the area burned in 1933 (as indicated in the author's notes) ranging from 10 to more than 25 ft, it is questionable whether check dams in the main channel, even of a very permanent nature, would have had any effect in reducing the magnitude of these flood waves. They might even have increased such magnitude by their retarding effect upon the advancing wave front. If such dams were placed in all of the side channels, and spaced so as to reduce the gradient therein by 50%, the velocity would have been reduced only 30%; but the velocity of surface runoff would not have been retarded. Therefore, the increased flow would have been ponded or stored in the smaller channels for a longer period of time but in all probability would have reached the main channels in flows of the same magnitude as it would have without such check dams.

Construction of check dams on side channels on, say, one half of the drainage area might have the effect of retarding the time of concentration from this part of the area, and thereby reduce main channel peaks by prolonging the outflow. Necessarily, such a program would require very careful planning and, in all probability, would be considerably more expensive than the present method of constructing debris basins below the mouths of the canyons and above the protected channels.

James M. Fox,[22] Jun. Am. Soc. C. E. (by letter).—The data of the notable New Year's floods of December 31, 1933, to January 1, 1934, in Southern California are utilized by Mr. Lynch in a manner quite different from that of other investigators in the same field. The conception of a momentary peak discharge far in excess of that which could be sustained by the maximum rate of precipitation should be of great interest to highway, railroad, and flood-control engineers. The most important lesson to be derived from the paper is that of anticipating such phenomena in the design of structures intersecting the channels of canyons such as those described by the author. In this respect, knowledge of the cross-sectional area occupied by the surge is far more essential than knowledge of the velocity or volume of the discharge. However, in neglecting to consider, fully, the effects of the heavy debris content on the formation of the flood peak, the author overlooked an opportunity to augment greatly the scope and interest of his paper.

As long as runoff and discharge are steady, the discharge rate remains at some value below the rate of precipitation. Obviously, in order to have a temporary peak created, in which the discharge is greater than that which could be sustained by the maximum rainfall intensity, a break in stream velocity must occur, by means of which slower moving water is overtaken by more rapidly flowing water. It does not appear invariably necessary, however, that this should be caused by a sudden increase in the rate of upstream runoff. The effect of an excessive peak could be produced as readily by a retardation of downstream velocities.

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[22] Staff Engr., Aluminum Co. of America, Vernon, Calif.

The statement is made that the debris content of the floods was from 50 to 70%. It is further stated that surface velocities were from 25 to 30 ft per sec, although the author confesses a lack of knowledge as to the velocities within the flow itself. Probably a mean velocity of 10 ft per sec across the section is not too low. If this were the case, the average peak discharge in the zones classified by the author under (*a*), (*b*), and (*c*), in Table 1, would have been 6,890, 2,830, and 1,740 cu ft per sec per sq mile, respectively, assuming only free water flowing. If one half of the areas were occupied by solid matter, however, the water runoff rate per square mile would then become 3,445, 1,415, and 870 cu ft per sec per sq mile. These values, although far more conservative than those obtained by using the author's velocities and cross-sectional areas as signifying the discharge of water only, are still large enough to lend support to the conception of excessive peak discharge. Apparently, the author took precaution, in his section measurements, to eliminate the effect of streambed erosion. It is presumed that these precautions were applied to care for possible increases in width, as well as depth, due to scouring.

In view of the large debris content of the floods, the writer is disposed to believe that excess cross-sectional area of the stream resulted from the velocity of the surge being greatly retarded, permitting the less-laden upstream runoff to overtake and add to the volume of the leading wave. All evidence available on the behavior of debris flows indicates that the surge travels with a "rolling" motion and with a high degree of turbulence. The corrosion of the channel is accomplished more by the violent vertical currents within the mass than by any great horizontal velocities. As fresh material from the channel is acquired by the fore part of the flow, tending to thicken the mass and retard its velocity, freer water from upstream, overriding the heavily loaded portion, overtakes and merges with the head of the surge, tending to reliquify it. This action may easily be simulated in miniature in the laboratory or in the field.

Mr. Lynch refers to the great floods of Northern Utah in 1923 and 1930. In the interest of advancing the understanding of these phenomena, the writer is impelled to enlarge somewhat upon the data supplied by the author, which, it is inferred, were obtained entirely from the references cited.

The writer has spent a year (1938–1939) on an investigation of cloudburst floods in Utah, under the direction of Ralf R. Woolley, M. Am. Soc. C. E., senior hydraulic engineer with the U. S. Geological Survey, at Salt Lake City. Three months of this period were passed in field exploration of some of the canyons involved in such floods, some of which are mentioned by the author. The remainder of the time was spent in the office on miscellaneous studies of these floods.

The storms producing the floods referred to by the author were entirely dissimilar to those causing the California New Year's disaster. No rain had fallen in the northern Utah areas for several days prior to the storms, and when they struck nearly all of the precipitation fell within the space of an hour.

The surges, or debris flows, accompanying these storms deposited on the fertile fans at the canyon mouths volumes of debris estimated at from 50,000 to 80,000 cu yd per sq mile of catchment area, practically all of which was removed from the channels themselves. In most cases there was but little runoff in advance of the great crest of the debris-charged flood. In some instances the material moving across the fans was so thick that its margins came to rest in banks 2 to 4 ft high, and with slopes of 30° to 40°, as if placed by a bulldozer.

Unfortunately, no accurate data could be uncovered as to the velocity or cross-sectional area of the flows. The streams explored by the writer had experienced two or more such floods in the 15 years since 1923. High-water marks had been obliterated by extensive undercutting and slumping of the high cut banks in the alluvium of the canyon bottom. In all probability the precipitation in the higher areas attained a rate of more than 4 in. per hr, for the space of a few minutes. It is possible that peak discharges in the main channel exceeded the maximum rates of surface runoff into the channel system. The effect of the high intensity of rainfall for the single, brief period was modified materially, no doubt, in the transit of the surface runoff down the mountain slopes and by absorption. However, the peaks in the main channel were the result of the accumulation of water from upstream behind what might be termed "moving dams" of debris-choked, muddy water.

It is stated by the author that the storm of August 13, 1923, in Northern Utah yielded less than 1.5 in. of rain. So far as has been found, the only direct recordings of rainfall in the area were obtained at stations of the U. S. Weather Bureau located near the foot of the western slope of the Wasatch Mountains, against which the storms impinged, and which are drained by the flooded canyons. These measurements showed total precipitation depths of from 0.64 to 1.34 in., depending upon their distance from the mountains and their location with respect to the storm center. The gages were all below an elevation of 4,500 ft, whereas the elevations of the catchment basins of the flood canyons ranged from 4,500 ft at the mouths to from 8,000 to 9,000 ft at the divide, 3 or 4 miles farther to the east.

Early in 1939 the writer made a rather extensive analysis of the variation of rainfall with altitude on the western slopes of the Wasatch Mountains, paying particular attention to the more violent summer thunder-storms. Based on the results of these studies, it is estimated that the depth of rainfall during the storms over the flood canyons varied from 1.25 to 2.50 in., depending upon the elevation, and averaged greater than 2.00 in. in depth over the entire area.

This greater volume of rainfall is confirmed also by the quantities of debris removed by the floods. Besides the writer's estimates, made in 1938, one of the references[6] cited by Mr. Lynch stated that in 1923, at Willard, Utah, 155 acres of land were covered by from 1 to 3 ft of rocks and earth. The minimum depth gives a total volume of 250,000 cu yd, or 62,500 cu yd from each of the 4 sq miles of the Willard Creek watershed. This is equivalent to three quarters of an inch in depth from the entire surface area. From several porosity determinations and shrinkage measurements it was found that at least an equal volume of water would have been required to liquify the debris mass, assuming that no water ran off beyond what was required to saturate the flow.

A glance at Fig. 9 will illustrate the improbability of a rain of less than 1.50 in., falling as recorded, accomplishing that amount of transportation. The intensities shown are for even 5-min intervals. The actual measured maximum intensity was 4.2 in. per hr (0.35 in. between 6:53 p.m. and 6:58 p.m.). The solid lines represent the intensities of the storm of August 13, 1923, as recorded by the recording rain gage of the U. S. Weather Bureau at Salt Lake City. Fig. 9 shows that if such a storm fell on ground whose dry infiltration rate was approximately 1 in. per hr, only about 0.5 in. would have been available for runoff.

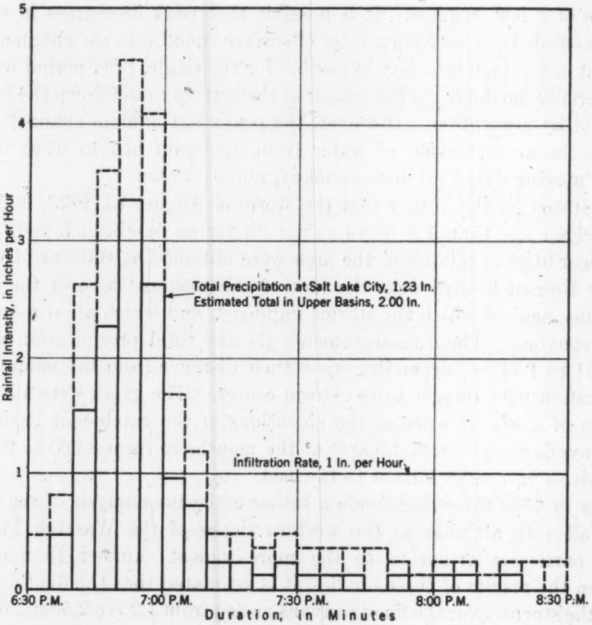

FIG. 9.—STORM OF AUGUST 13, 1923, AS RECORDED AT SALT LAKE CITY, UTAH

The dotted outline (the average precipitation over the basins, August 13, 1923) is based upon the writer's estimate of 2 in. in depth. It was assumed that both duration and intensities were increased somewhat by the effect of the rising terrain over which the storm was passing. If the rain fell in a manner similar to that indicated, and an infiltration rate of 1 in. per hr is again assumed, about 1 in. would have run off in the course of half an hour. How much more of the rain would have been absorbed in transit over the slopes is a matter of conjecture. Moreover, if the various determinations of debris volumes are reasonably correct, the writer's estimates of rainfall and intensities are more likely to be low than high.

F. C. FINKLE,[23] ESQ. (by letter).—Before discussing the author's hypothesis, it should be clearly understood that the type of floods to which it is applicable is not the same as that resulting from what are called "cloudbursts" in the Western States. Cloudbursts are not preceded by any rainfall that can be regarded as initial. On the contrary, the ground is dry before precipitation begins; the downpour is most rapid at the start and, after exhausting itself within a short space of time, slackens to a slow drizzle of short duration, or stops abruptly. Although little has been learned concerning the rate of precipitation causing cloudbursts, some facts have been recorded indicating rates varying from 10 to 20 in. per hr, but rarely, if ever, lasting for nearly one hour. The writer observed an instance in the Crafton Hills near Redlands, Calif., during August, 1891, in which 8 in. of water fell in 37 min, starting without any previous fall and ending without tapering off perceptibly. He also observed another during the summer of 1903 in a small canyon east of Mojave, Calif., in which 5.5 in. fell in 22 min.

These and similar downpours on a dry surface result in practically a 100% runoff in a short time and produce a nearly vertical wall of onrushing water in the drainage channel. These torrents are entirely consistent with the total fall of water within a given time, because there is even less absorption than the volume of mud and debris picked up and carried by the water, which compensates for the penetration loss. The cloudburst falling on a hot, dry, and dusty surface causes the first globules of water to become encased in a film akin to the spheroidal film around a drop of water thrown upon a hot stove, thus preventing any considerable penetration. The rapidly descending volume is also retarded by debris and surface irregularities, until these are absorbed into the water. In this manner, both lack of penetration and checking of flow in the early stage of downpour result in piling up the traditional wall of water.

The so-called Montrose flood near Glendale, Calif., which reached its peak near midnight of December 31, 1933, and January 1, 1934, was in no manner comparable to a western cloudburst. It was the culmination of a sustained foothill and mountain rainfall commencing on December 30. It was the second most severe downpour and flood witnessed by the writer in this same locality, being surpassed only by the previous one of December 24–25, 1889, at which time there were no devices for measuring precipitation intensities at any point in Southern California. However, the much greater volume of debris in the earlier flood, and the greater distances to which it was carried, as well as the higher flood peaks, in spite of the less impervious character of the watershed surface, all indicate that flood to have resulted from an intensive precipitation more than twice as great in a single hour as the highest for December, 1933. The only data available on the 1889 intensity are two rough observations made near Westlake Park in Los Angeles for the hour from midnight of December 24 to 1:00 a.m. of December 25, 1889, in tin cans, one by the writer showing 2.89 in. and the other by Burr Bassell, showing 2.75 in.

During the 1933–1934 Montrose flood, the writer resided in the City of Burbank, Calif., which is a short distance from the foot of the Verdugo Mountains, southerly from Stough Canyon and southwesterly from Sunset Canyon in the Verdugo Mountains. He spent both days, December 31, 1933, and January 1, 1934, at home because of the heavy downpour and dangerous condition of the streets. He made several trips, during the afternoon of December 31 and up to 10:00 p.m., as far east as the Glendale or Verdugo wash, observing the constantly rising flood water, after which all the streams from Stough Canyon to Verdugo wash became impassable.

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[23] Cons. Engr. and Geologist, Burbank, Calif.

No professional visit to the area covered by the Montrose flood was made by the writer until January 30, when engineering observations and studies were made from Haines Canyon on the west to Hall-Beckley Canyon on the east. The Pickens Canyon data were the only ones fully worked up and analyzed by the writer. The best section found in this canyon was near the Gould Castle with a good stretch of uniform channel, not affected by remnants of partly destroyed check dams, and with but shallow deposition after the flood peak. The hydraulic properties of this stretch were as follows: $a = 572.9$ sq ft; $p = 71$ ft; $r = 8.07$ ft; and $s = 0.09$. Data, therefore, were secured from cloudburst floods, saturated with silt, sand, gravel, and other debris passing over the diverting dam at Mill Creek No. 3 hydroelectric plant, near Forest Home, Calif., where accurate gagings of summer cloudburst floods had been made. On the basis of these data the value of n in Kutter's formula was placed at 0.07 for calculating discharge, which gave 16,267 cu ft per sec.

This peak flow was far from being water that had descended on the 1.78 sq miles of mountain drainage above the station. The remnants or locations, where they had existed, of more than 500 check dams, which had been constructed by the Los Angeles County Flood Control District of loose rocks and gravel wrapped in wire netting, were examined, and their contents in the washed out sections and storage behind them were measured and calculated. This volume of about 3,300,000 cu ft, about one half water and one half silt, sand, gravel, and rocks was added to the peak flood.

Next, by means of standard formulas applicable to a small drainage such as Pickens Canyon, it was determined that, with the rainfall coming as shown by the Los Angeles County Flood Control rain gages, the peak flood should have been about 5,300 cu ft per sec in water, taking into account the manner of concentration and the pyramiding of the runoff from the intensive period upon the channel flow preceding. Without considering the debris gathered by this 5,300 peak, there were nearly 11,000 cu ft per sec of the measured peak to account for. This differential must be accounted for by the water and debris from the failed check dams, other debris picked up by moving water on the slopes and in the channels, or on the theory advanced by the author.

Obviously, the volume in cubic feet per second resulting from the obliterated check dams would depend upon the duration of the peak, after reaching its maximum and before it began its rapid decline. The only data on duration of maximum peak came from those who saw the flood from the banks of Pickens Canyon at points below Gould Castle. The few critical observers interviewed differed widely, ranging from 5 min to about a quarter of an hour. From these statements, the writer reached the conclusion that the sustained maximum peak did not exceed 9 min, which would cause the water and debris from the check dams to have added 6,000 cu ft per sec to the peak flood, on the assumption that the check dams failed during the time of peak. The writer does not wish to force any of his own opinions on others, but desires to say that he reached the firm conclusion that the check dams went out with the first onrush of the peak flow near the head of the canyons, and were washed out one after the other as the peak traveled downstream. If it happened in this manner, not only would 6,000 cu ft per sec of water and debris be added to the volume, but the momentary delay, as one check dam after another was hit, would accumulate volume sufficient to compensate for channel storage and to raise the peak flow progressively.

If the foregoing solutions are accepted (as they are by the writer), there are less than 5,000 cu ft per sec of the measured peak to be accounted for, which must have consisted of solid material such as ashes, silt, sand, gravel, rocks, and boulders from the slopes and channels; or, from the author's hypothesis, resulted from hydraulic surges in the channel flow itself, as illustrated in Fig. 1. If the debris content was about 50% of the volume, as pointed out by the author under "Debris Content," and writer's solution, as given herein, explains the great volume of the peaks; and the surge theory is unnecessary to account for runoff in excess of rainfall, as the solid content is computed to be less than 50%.

That channel surges occur in flood channels during protracted rain storms is a well-established fact. They may result from an "hydraulic jump" in the channel, in which case their effect is slight, or from a marked increase in the rate or intensity of precipitation during a storm, as pointed out by the author, in which case the effect is violent, because the resulting higher channel velocities cause the later discharge to overtake the lesser previous one. Surges also result from overloading a stream with debris, resulting in a pause of varying duration, in which case the stream volume may become greatly augmented by reason of the interruption to its uniform progress.

In the Montrose flood, under discussion herein, the increase in rainfall intensities was a minor factor in causing surges and radical augmentation of stream volume. In fact, as the author remarks (see heading "Verdugo Mountains: Rainfall on January 1, 1934): "There were no noteworthy short-time rainfall intensities." Thus, the cause being nearly absent, the effect was absent also; but other causes were present, which accounted for the noteworthy surge phenomena in that flood, and are stated in the order of their importance, as follows: (1) The great number of flimsily built check dams, which failed during the flood; (2) the loosened rock material and ashes caused by recent burning over of the drainage areas; and (3) debris collected from the bottom of the stream channels, due to the numerous small waterfalls over the check dams before they failed; these disrupted the stream beds and changed their regimen.

Although the author ignores the effect of the check dams, he states (see "Flood of January 1, 1934"): "The greatest surges were those in the then recently burned area." This statement agrees with observed fact, but is incomplete without including the check dams in channels within the then recently burned area.

Other studies made by the writer in Haines, Blanchard, Dunsmuir, and Hall-Beckley canyons, etc., bear out the foregoing statements, so far as the data have been analyzed; but these analyses do not go beyond computing the peak volumes, and detailed data on other matters are lacking. A study was also made of the peak in Verdugo Canyon, where all the streams from the Montrose-Crescenta area concentrate, but the results do not bear on the question considered herein, because of channel and flood-plane regulation and varying time for synchronizing the flows below the foot of the Sierra Madre.

A. L. Sonderegger,[24] **M. Am. Soc. C. E.** (by letter).—Large areas of the choice foothill lands of Southern California occupy the coalescent debris cones of streams which issue from high mountain ranges. A clear conception of the phenomena which accompany the torrential floods of these streams is essential to the planning of flood protection works. Mr. Lynch's paper contributes information that is valuable to the analysis of the problems presenting themselves to flood control engineers.

Apparently, the conclusion to be drawn from the paper is that "floods of the so-called 'cloudburst' type yield momentary runoff peaks entirely out of proportion to the rate of rainfall." Since this conclusion is principally based upon data and records pertaining to the storm of December 31, 1933, in La Crescenta and La Cañada valleys, a foothill area near Pasadena, it may be appropriate to review some of the conditions surrounding this event. This was an important flood in that it led to the adoption, in Los Angeles County, of debris basins for the control of the floods of minor steep watersheds, and to the abandonment of loose rock wire-wrapped check dams as a major flood control measure.

Reference to the flood of January 1, 1934, in the same locality, was made by the writer in 1935.[25]

The individual watersheds contributary to the area are small—less than 2 sq miles. The mountainous sections consist of relatively steep slopes, covered with a veneer of residual granitic soils, generally 1 to 3 ft in depth. Creek channels are deep incisions which, in the long-period intervals between heavy storms, accumulate large quantities of debris, but are scoured clean during major floods. Most of the creeks are entrenched in an ancient alluvial mesa extending from mouths of canyons to the debris cone. The disaster affected the settlements on the modern cone area. A major portion of the watershed was burned in November, 1933. As a measure of protection to the residential settlements, the channels of creeks were stepped down by the construction of check dams, beginning at the lower mountainous section and extending on to the debris cone as far as the State Highway. These checks were wire-wrapped boulder rolls about 4 ft high and 150 ft or more apart.

No major flood had occurred in this area for about 17 years and creek channels showed a corresponding effect of an accumulation of debris. Moreover, the check dams had formed a succession of small basins which would intercept debris rolling in from steep slopes, or carried by the streams. Therefore, the "stage was set" for sheet floods from steep burned watersheds, a preliminary storage of debris of large proportions behind the checks, and a subsequent removal of detritus, new and old, when runoff had assumed such proportions as would set the alluvial mass into motion.

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[24] Cons. Engr., Los Angeles, Calif.

[25] "Modifying the Physiographical Balance by Conservation Measures," by A. L. Sonderegger, *Transactions*, Am. Soc. C. E., Vol. 100 (1935), p. 284; also "Flood in La Cañada Valley, California," by H. C. Troxell, Assoc. M. Am. Soc. C. E., and John Q. Peterson, U. S. Geological Survey *Water Supply Paper No. 796–C*, 1937.

The major residential settlement of the area of coalescent cones occurred during the period from 1917 to 1933. The disaster occurred on January 1, between 12:00 and 12:30 a.m.

The accumulated rainfall for the storm of December 31, 4 a.m. to 12 noon, is as follows:

Area	Rainfall, in inches	Elevation, in feet
Flintridge	10.5	1,325
Sister Elsie Peak	7.8	4,950
Haines Canyon	9.0±	3,470
Pasadena	9.65	865

The greatest rise in 5 min intensity at Sister Elsie Peak was at 11:40 p.m. from a rate of 0.60 to 1.56 in.

The maximum intensities in the valley station of Flintridge were as shown in Table 5(*a*). These are not really excessive rates and are far below cloudburst proportions. For example, in the heavy rain of January 8, 1940, the Upper Haines gage at the westerly margin of La Crescenta Valley furnished the record given in Table 5(*b*). The conclusion reached is that the major cause of the disaster was the saturation of the denuded slopes by an accumulated rain of 9 in. within a period of 20 hr, and a final increase in intensity from 0.60 to 1.56 in. for a 10-min. duration.

TABLE 5.—COMPARISON OF MAXIMUM RAINFALL INTENSITIES

Time period, in minutes	(*a*) FLINTRIDGE FIRE STATION, DECEMBER 31, 1933		(*b*) UPPER HAINES GAGE, JANUARY 8, 1940	
	Rainfall, in inches	Intensity, in inches per hour	Rainfall, in inches	Intensity, in inches per hour
4	....	....	0.62	9.30
5	0.24	2.88	....	....
10	0.34	2.04	0.80	4.80
20	0.60	1.80	1.00	3.00
30	0.84	1.68	1.27	2.54
60	1.33	1.33	1.37	1.37
120	2.14	1.07	....	....

Although an accumulated rain of 9 in. exceeds that normally occurring during capital floods in this area, the rate of maximum intensity has frequently been greatly exceeded. The conclusion is that other combinations of accumulated rain and intensities on similarly located burned-over watersheds are conceivable, which may produce more severe flood conditions.

Peak Flows.—Certain facts and conclusions become apparent from a study of Table 1:

1. Cross sectional areas of peak flows of recently (November, 1933) burned-over slopes average 689 sq ft per sq mile as compared with 283 sq ft of slopes

burned in 1927, and 174 sq ft of unburned small canyon watersheds. In estimating probable flood peak flows, the conditions imposed by a recent burn, therefore, should form the basis of assumptions.

2. The cross-sectional areas of peak flows per square mile of burned watersheds show widely different values. Since the factors that affect runoff, such as the state of the watershed cover, slopes, orographic conditions, and rainfall, were very much alike in the adjacent burned-over drainages and would call for more uniformity in wetted channel sections, it is apparent that causes other than those normally present were responsible for such erratic values.

Abnormal conditions may have resulted from sheet floods, landslides, log and boulder jams, failure of check dams, and scour.

The existence of check dams would result in a high-water mark during the first stage of the flood flow. Checks would act as debris barriers and would become completely backfilled. After the collapse of checks, an enormous viscuous mass of debris would be subject to removal, resulting in a deep scour of the creek beds and new high-water marks, but at lower elevations. The resulting cross sections, measured after the flood, therefore, would be representative of peak flows at two different stages, the high-water mark resulting from the earlier peak and the scoured bottom from the later one. The inference is that these cross sections, combined, gave values too large and not representative of any stage of the flood; the corollary is that runoff estimates based thereon would be misleading.

The composition of the fluvial mass removed during the flood was apparent from deposits above the abutments of check dams. It was a heterogeneous mass of unassorted material resembling, in composition, the well-known mud flows caused by cloudbursts in desert areas.

The numerous scars visible on the mountain slopes after the flood pointed to land slides of saturated residual soil cover, shoestring and gully formation, and bare areas, were proof of sheet floods carrying a heavy percentage of silt and debris at the time of peak flow. Indications are that the bulking of the viscuous mass may have been in the proportion of debris to water of 2 : 1 and as much as 3 : 1.

In March, 1934, engineers of the Los Angeles County Flood Control District measured five peak flow cross sections of Pickens Canyon, all within a distance of 140 ft, the tributary drainage being 1.74 sq miles. The areas per square mile were as shown in Table 6. These values check closely with the author's cross sections C-3 to C-5 (Table 1(*a*)).

TABLE 6.—Cross Sectional Areas, Pickens Canyon, March, 1934
(Catchment Area, *M*, Equals 1.75 Sq Miles)

Description	Sections					Mean
	a	*b*	*c*	*d*	*e*	
Cross-sectional area, in square feet	400	705	602	727	565	600
Hydraulic radius, *R*, in feet	7.22	10.22	9.85	9.70	8.44	....
Area, in square feet per square mile (direct proportion)	229	400	344	415	323	342

Under conditions prevailing in these small watersheds during the flood, the computation of velocities with formulas by Kutter or Manning would lead to erroneous results. These formulas apply to the flow of water, whereas the runoff of the New Year's flood was a viscuous mass of unassorted debris. The writer knows of no experimental determination of "*n*" with bulking of 2 : 1 or 3 : 1.

No information is available as to the time and manner of the collapse of the numerous check dams. Suffice it to say that after the flood the canyons were fairly clear of debris and of checks.

Evidence was found of high velocities in all canyons. These have been computed by the author from the location of splashes from the muddy water, at 25 ft per sec; but these splashes may have occurred prior to the removal of checks. There were also indications of a protracted heavily loaded peak flow evidenced by tree trunks 8 in. or more in diameter, which, at points some distance above the stream bed, had been abraded to a depth of several inches.

Estimates of average velocities during the peak of flood were made by the writer, based upon circumstantial evidence. Reports of eyewitnesses indicate that the destructive peak flow arrived in Montrose, Calif., about 15 min after leaving the mouth of canyons. The period of intense rainfall started at 11:45 p.m. on December 31, whereas the peak flow reached Montrose between 12 and 12:15 a.m. on January 1.

Summarizing available data, it was concluded that the average velocity of the peak flow of the bulked material was 10 to 15 ft per sec, and that the proportion of debris to water may have varied from 1 : 1 to 2½ : 1, or even higher.

It is believed that the mud stream remained intact as a highly viscuous mass from its inception in the canyon and through the ravine below the mouth and down to the debris cone where it was dispersed by obstructions in its path. After this mass came to rest, the water drained away and found its way to the intercepting channel of Verdugo Creek. It is probable that in the canyons and ravines, during this phase of the flood, a discharge of free water occurred at higher velocities on top of the mud stream and more or less separated therefrom. These would account for waves and walls of water observed by witnesses.

On the basis of estimated mean peak flow velocities of 10 to 15 ft, cross sections measured in Pickens Canyon, with a drainage area of 1.75 to 1.80 sq miles, the discharge given in Table 7 is obtained. These are high discharge

TABLE 7.—COMPARISON OF DISCHARGE QUANTITIES, PICKENS CANYON

Description	AUTHOR'S ESTIMATE		WRITER'S ESTIMATE	
Average velocities, in feet per second	10	15	10	15
Catchment area, *M*, in square miles	1.78 to 1.80		1.75	
Cross-sectional area, in square feet	641*		600†	
Discharge, in cubic feet per second	6,410	9,615	6,000	9,000

* Mean of Sections C-3, C-4, and C-5, Table 1. † Mean of Sections *a*, *b*, *c*, *d*, and *e*, Table 6.

values, even for short transient peaks; they fall on or outside the enveloping curves developed by different writers.

In 1925, C. S. Jarvis, M. Am. Soc. C. E., reported data[26] from which it appears that, for 2 sq miles of watershed, $Q_{max} = 7{,}000$ cu ft per sec per sq mile. Assuming the watershed of Pickens Canyon, tributary to the peak cross section given in Table 7, to average 1.78 sq mile, the Jarvis enveloping maximum would be, for Pickens Canyon, $Q = 7{,}000\sqrt{1.78} = 9{,}350$ cu ft per sec.

In 1939, William P. Creager,[27] M. Am. Soc. C. E., shows an enveloping curve for that year which, for watersheds of 2 sq miles, gives a flood peak of 4,000 cu ft per sec per sq mile. Applying this value to Pickens Canyon, and on the basis of direct proportion to the watershed, the probable peak flow would be less than a value of $1.78 \times 4{,}000 = 7{,}120$ cu ft per sec.

In order to hold estimates of Pickens Canyon peak flow within the aforementioned enveloping curves, the computer must either assume velocities of less than 15 ft per sec, or smaller cross-sectional areas. It is believed that the latter assumption gives results more consistent with the observed nature of the peak flow and its disastrous effects.

Volume of Debris Removed.—Surveys of Pickens Canyon before and after the flood permit approximate computations of removed debris. Mr. Troxell[25] estimated this volume at 72,000 cu yd whereas the writer estimated it at 98,000 cu yd. The latter is at the rate of $\dfrac{98{,}000}{\sqrt{1.78}} = 73{,}000$ cu yd per sq mile.

The Los Angeles County Flood Control District has adopted the plan of locating debris storage basins at mouths of canyons of the Sierra Madre. These basins are cleared after each flood. The required storage capacity is a function of the following factors: The debris discharge of a major flood; the interval between the floods of a season; and the capacity of the excavating equipment that is available for the clearing of a basin immediately after a flood.

TABLE 8.—ANALYSIS OF THE RECORD CRITICAL FLOOD YEAR

Date	Precipitation*	Flood character	Remarks
October 23, 1889.......	3.62	Moderate	Washouts on railroads; unprecedented rainfall for this time of year; streets flooded
December 12–15, 1889..	4.30	Heavy	Traffic paralyzed by washouts; streets flooded
December 25, 1889.....	3.82	Heavy	Flood scarcely less severe and disastrous than that of 1884; harbor silted
January 26, 1890.......	4.17	Heavy	Streets converted to rivers; traffic suspended; harbor silted

* Precipitation in 24 hr, in inches.

The critical flood year of record is believed to be that of 1889–1890, for which Ford A. Carpenter has analyzed flood conditions as shown in Table 8.[28]

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[26] "Flood Flow Characteristics," by C. S. Jarvis, *Transactions*, Am. Soc. C. E., Vol. 89 (1926), Plate IX, facing p. 994 (see enveloping curve marked "$Q = 10000\sqrt{M}$; modified Myers maximum 100%").

[27] "Possible and Probable Future Floods," by William P. Creager, *Civil Engineering*, November, 1939, p. 670, Fig. 6.

[28] "Flood Studies at Los Angeles," by Ford A. Carpenter, *Monthly Weather Review*, Washington, June, 1914, 42: 385–391.

The occurrence of three major floods within a period of 45 days makes the feasibility of clearing a series of basins after each flood problematical. It is reasonable to assume that debris discharge would be a maximum during the first flood and less with each succeeding storm. However, it is believed that under the conditions prevailing in the foothills of Southern California a basin capacity of 300,000 cu yd per sq mile should be provided.

Check Dams.—The failure of hundreds, possibly thousands, of check dams during the flood of January, 1934, was responsible for the Resolution passed by a conference of engineers, foresters, agriculturists, and investigators, held under the auspices of the Los Angeles Section of the Society, in 1935, and reported in the Progress Report of the Committee of the Irrigation Division on "Conservation of Water."[29]

In its five conclusions the Conference expressed its disapproval of check dams for the regulation of capital floods and advocated permanent debris dams in mountain watersheds, properly designed and constructed. It is believed that the substance of these conclusions has generally been accepted, at least in Southern California, and that check dams as a major flood control measure are no longer built in the steep mountain watersheds.

HAROLD C. TROXELL[30] AND R. STANLEY LORD,[31] ASSOC. MEMBERS, AM. SOC. C. E. (by letter).—Shortly after the very severe storm of January 1, 1934, the writers were called upon to make an investigation and prepare a flood report[19] for the U. S. Geological Survey. After a thorough inspection of the area it was apparent that the flood flows resulting from this storm were entirely unlike those that ordinarily occur in this area. Unusually large cross sections, similar to those shown in Table 1, were found, and the writers' explanation was given to account for the large sections. On the basis of the evidence found in the area, the explanation made at that time seems more plausible than that indicated by the author.

As the phenomenon experienced during this storm was more or less similar in all the canyons draining the burned-over area tributary to the La Cañada Valley, the investigation was confined mainly to Pickens Canyon, which is considered typical. Numerous check dams had been built in the stream beds of practically all of the streams in this area. A typical view is shown in Fig. 10. In Pickens Canyon, 182 check dams had been constructed in the 17,000 ft of main channel lying between the 1,550-ft and 4,000-ft contours, and additional check dams had been built in the tributaries. The stream-bed slope, in this 17,000 ft of main channel, varies from 7% at the lower end to about 37% at the upper end. The check dams averaged between 5 and 10 ft in height and were instrumental in reducing, to a certain extent, the effective slope of the channel. This tended to produce slower velocities, causing deposition of debris above each of these structures. However, at times of extreme flood the slope tends to return to the normal slope of the stream channel. The deposition of material above each of these structures tends to increase the velocities, due to smoothing of the channel; but this tendency is more than offset by the obstruction caused by the heavily rooted plant life that develops in the deposits.

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[29] *Proceedings*, Am. Soc. C. E., December, 1935, p. 1490.

[30] Associate Hydr. Engr., Water Resources Branch, U. S. Geological Survey, Los Angeles, Calif.

[31] Associate Hydr. Engr., Water Resources Branch, U. S. Geological Survey, Harrisburg, Pa.

Pickens Creek flows in a channel carved in bedrock above about the 2,000-ft contour and in a channel that is cut through the debris cone below that point. An inspection of the bedrock sections showed that the check dams, together with the debris deposits above these structures, had been completely washed out of the channel during the New Year's flood. Profiles obtained before and after this flood showed that, for a distance of 6,200 ft upstream from the 2,100-ft contour, the channel had been lowered as much as 15 ft in some sections, with an average lowering of 6.5 ft. Practically all of the debris that had accumulated in the canyon during a period of several years of relatively light runoff was removed from the canyon in this single flood. Based on these two profiles it was estimated that about 72,000 cu yd of stream-bed deposits were removed from the canyon sections of this 1.6-sq-mile drainage area and deposited on the valley below.

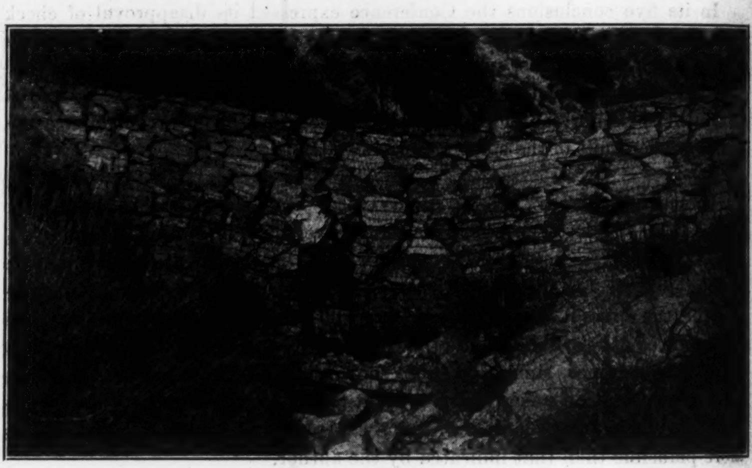

FIG. 10.—TYPICAL CHECK DAM

Courtlandt Eaton,[32] M. Am. Soc. C. E., has shown a typical set of four adjacent cross sections taken in Pickens Canyon after the New Year's storm. He further states[33] that, although the highest water marks were made when the debris flows were moving very slowly, or not at all, the final lowest channel sections were caused by the later and receding stream flows of higher velocities.

In discussing Mr. Eaton's paper, E. I. Kotok and C. J. Kraebel[34] show a photograph of an additional section in Pickens Canyon. This cross section, together with the sections by Mr. Eaton, compare quite favorably with those given in Table 1. As indicated by Mr. Eaton, it is contended that these large cross sections are largely the result of a debris movement.

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[32] "Flood and Erosion Control Problems and Their Solution," by Courtlandt Eaton, *Transactions*, Am. Soc. C. E., Vol. 101 (1936), p. 1325.

[33] *Loc. cit.*, p. 1324.

[34] *Loc. cit.*, Fig. 15, p. 1352.

As a result of his study on debris movement G. K. Gilbert[35] states that if a stream, loaded to its full capacity, reaches a point at which the slope is less, it becomes overloaded with reference to the gentler slope and part of the load is deposited. If a fully loaded stream reaches a point where the slope is steeper, its enlarged capacity causes it to take more load which, in turn, erodes the bed. If the slope of a stream bed is not adjusted to the discharge and to the load it must carry, "the stream continues to erode or deposit, or both, until an adjustment has been effected and the slope is just adequate for the work."

Thus Nature attempts to maintain a balance between slope, debris load, and discharge. Canyons such as Pickens are very irregular in alinement, cross section, roughness, and slope, with the result that the capacity of a given discharge to carry debris varies considerably from point to point. For example, an exceedingly rough section, a constricting section, or a sharp bend in alinement might cause considerable loss in velocity. With this loss in velocity there is a corresponding loss, but of a much greater magnitude, in the capacity of flow to transport debris; and, as a result, if the stream is loaded to capacity, deposition occurs at or above these controlling points and will continue until the slope is increased or friction factor reduced to such a point that the transporting capacity of the stream is equal to the capacity in the sections upstream. Equilibrium, once established, will continue as long as the debris load and rate of discharge remain constant. With a runoff as unstable as that occurring during the New Year's flood, periods of equilibrium would be of a very short duration.

For many years prior to this storm Pickens Creek had discharged practically clear water on the debris cone. Consequently, there had been a tendency to maintain, upstream from the cone, the flattest possible stream-bed slope that the alinement of the canyon and the roughness of the channel would permit. As stated elsewhere,[36] in a deep, irregular canyon, such as that of Pickens Creek, the normal balancing of debris deposits and channel slopes that might be suitable for the movement of the usual small debris loads develops a channel that is unsuitable and out of balance for the infrequent large flows with their attendant debris movements. With the continuous and heavy rainfall that characterized the New Year's storm, much debris was delivered to the stream channel. In order to transport this abnormally large debris load, the stream had to increase its transporting capacity, either by smoothing its bed or increasing its gradient, or by a combination of the two processes.

The rainfall characteristics of the New Year's storm were such that there were a number of sharp peak runoff periods prior to the midnight storm. Each of these periods of runoff had a tendency to build up and scour out the debris deposits above each of the controlling sections in the channel. The net result of these operations was the gradual movement of the debris load down the canyon to the flatter and broader sections of the debris cone. G. H. Cecil[37] has reported local observers to the effect that the early runoff transported relatively little mud and silt. According to Mr. Cecil, the first reported loss of life occurred in Montrose at about 9 p.m. (December 31, 1933), at which time the flood was said to have been comparatively free of debris.

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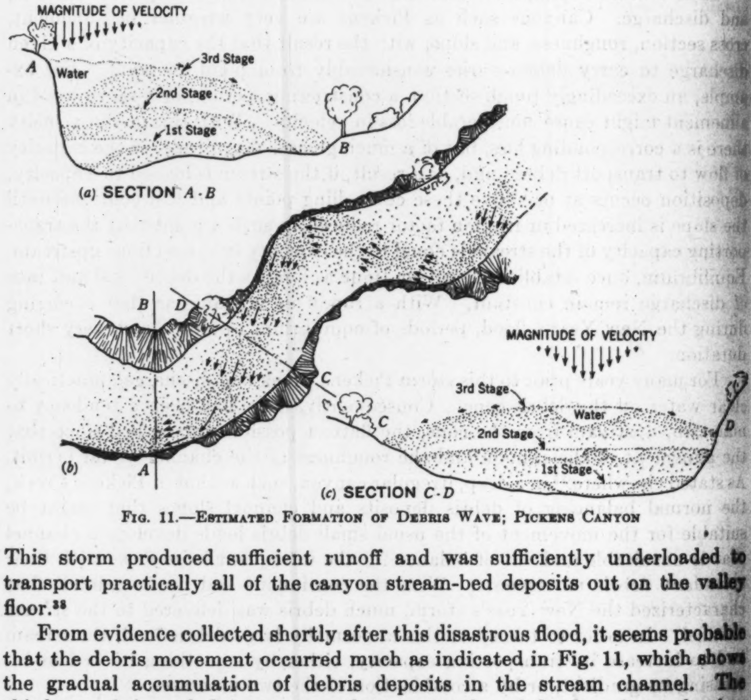
[35] "Hydraulic-Mining Debris in the Sierra Nevada," by G. K. Gilbert, *Professional Paper No. 105*, U. S. Geological Survey, 1917, p. 26.

[36] "Flood in La Cañada Valley, California, January 1, 1934," by Harold C. Troxell and John Q. Peterson, U. S. Geological Survey *Water Supply Paper No. 796-C*, 1937, p. 77.

Thus, after almost 12 hr of continuous rainfall, there was little movement of debris across the debris cone; apparently it was stored in the lower reaches of the canyon section or the upper part of the debris cone. From available information it appears that practically all of the debris was moved across the debris cone and deposited in La Cañada Valley as a result of a very heavy shower or downpour that occurred just before midnight on December 31, 1933.



(a) SECTION A·B

(b)

(c) SECTION C·D

FIG. 11.—ESTIMATED FORMATION OF DEBRIS WAVE; PICKENS CANYON

This storm produced sufficient runoff and was sufficiently underloaded to transport practically all of the canyon stream-bed deposits out on the valley floor.[38]

From evidence collected shortly after this disastrous flood, it seems probable that the debris movement occurred much as indicated in Fig. 11, which shows the gradual accumulation of debris deposits in the stream channel. The third stage is intended to represent the cross section of the deposits at the time the high-water marks were made. It appears evident that these high-water marks may or may not have been made at the time of maximum discharge.

At points in the channel, such as section *A-B*, Fig. 11, the main thread of the flood is trained by the section immediately upstream so that it tends to hug the outside of the bend. As deposition must take place along the boundary between the moving water and debris, and debris at rest, there is a gradual

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[37] "Fire and Flood," by G. H. Cecil, Conservation Association, Los Angeles County, 1934.

[38] "Flood in La Cañada Valley, California, January 1, 1934," by Harold C. Troxell and John Q Peterson, U. S. Geological Survey *Water Supply Paper No. 796-C*, 1937, p. 83.

FIG. 12.—EAST BANK OF PICKENS CREEK AT ORANGE AVENUE; MATERIAL FROM THE DEBRIS WAVE PASSED OVER THE TOP OF THIS 23-FT BANK

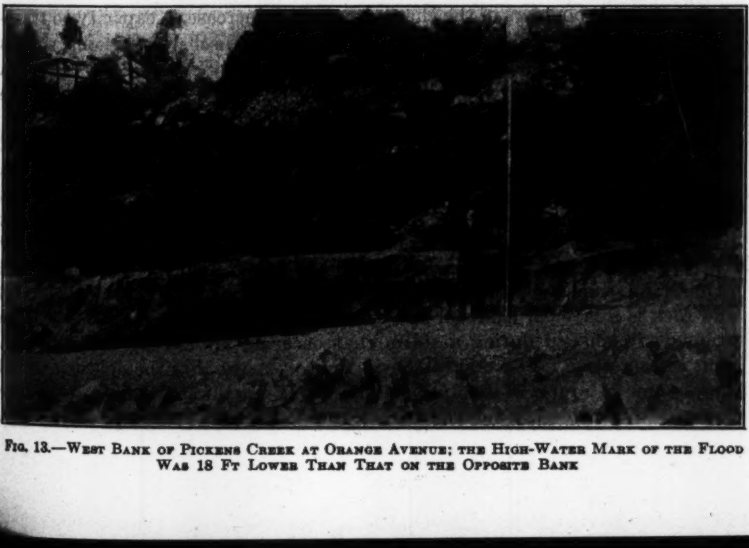

FIG. 13.—WEST BANK OF PICKENS CREEK AT ORANGE AVENUE; THE HIGH-WATER MARK OF THE FLOOD WAS 18 FT LOWER THAN THAT ON THE OPPOSITE BANK

building up of the deposit on the outside of the bend. For example, near the end of Orange Avenue in Pickens Canyon, a section was found very similar to that indicated by section *A–B*, in which the high-water marks indicated a difference of 18 ft between the bank elevations on the two sides of a channel less than 70 ft wide. Figs. 12 and 13 show both the east and west banks of this section. The high-water marks and debris deposits on the top of the east bank were 23 ft above the bed of the stream when the photograph was taken. On the other bank the high-water marks are only about 3 ft above the bed of the stream. It has been suggested by the author that this difference in elevation might be the result of superelevation. However, the composition of the banks of the channel was such that it seems quite unlikely that they could have withstood the eroding action of the stream flow if the velocity had been sufficient to produce a superelevation of this magnitude.

Fig. 14 shows the marking on the outside wall of a bend in the channel. This point is comparable to the point marked *A* in Fig. 11(*a*). Note how the rapidly moving water and debris have scoured the bank smooth; and note also the lack of channel erosion. Thus the stream flow appears to have been loaded to capacity at the time these marks were made; otherwise the channel would have been badly undercut and eroded. Figs. 12 and 13 would seem to indicate that not until the final stages of the debris movement was the runoff sufficiently underloaded to start the erosion process.

The general longitudinal section of the debris movement is shown in Fig. 16. The length of this section is sufficient to represent the time of passage at any given point of the major part of the debris movement. At a section such as that marked *C–D* the stream flow has the maximum transporting capacity. Because of poor flow conditions, such as flatter slopes and the distribution of the flow into a number of channels, a part of this capacity is lost before reaching section *A–B*, with the result that deposition takes place. The downstream movement of the debris is obtained by the scouring effect at the upper section and its movement to points farther downstream. Fig. 15 shows a section such as *C–D* in Fig. 16. It should be noted how the natural process of debris deposition among boulders in the channel favors an increase in capacity to transport debris (see Fig. 16). At the time of the peak discharge this deposit was probably almost level across, with a shallow, fast-moving, heavily debris-laden stream on its surface. Subsequently a smaller, but underloaded, discharge had removed a large part of the deposit.

Sections of debris waves, like the one indicated in Fig. 16, were entrapped at numerous places along the channel, due to irregularities in alinement. Such a section is shown in Fig. 17. The upper half of this figure shows the deposit left by the debris wave. A close inspection will show a marked contrast between the debris-wave deposit and gravel and rock deposit immediately below it. The gravel and rock deposit, composed in the main of recently eroded material, is remarkably free from fine particles. Immediately below the gravel and rock is a layer of charcoal and ash which was probably deposited on the original stream bed during a storm that occurred on December 13 and 14. This is interesting for several reasons. First, it shows that the debris material was composed largely of the finer materials. It was the presence of these finer particles that made the movement indicated in Figs. 11 and 16 possible. Mr.

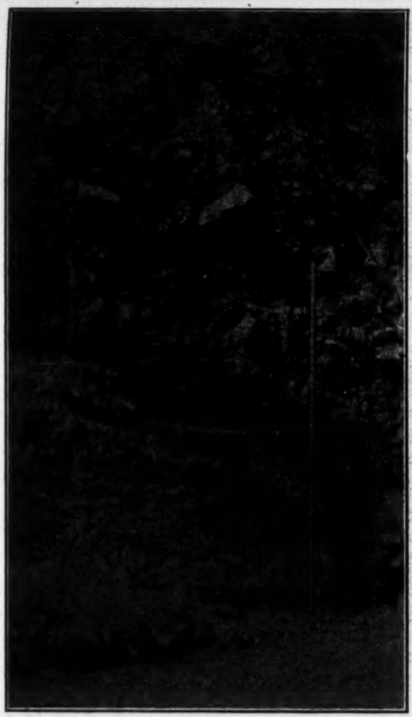

Fig. 14.—Scour of Side Wall in Pickens Creek Channel. Note the Lack of Erosion

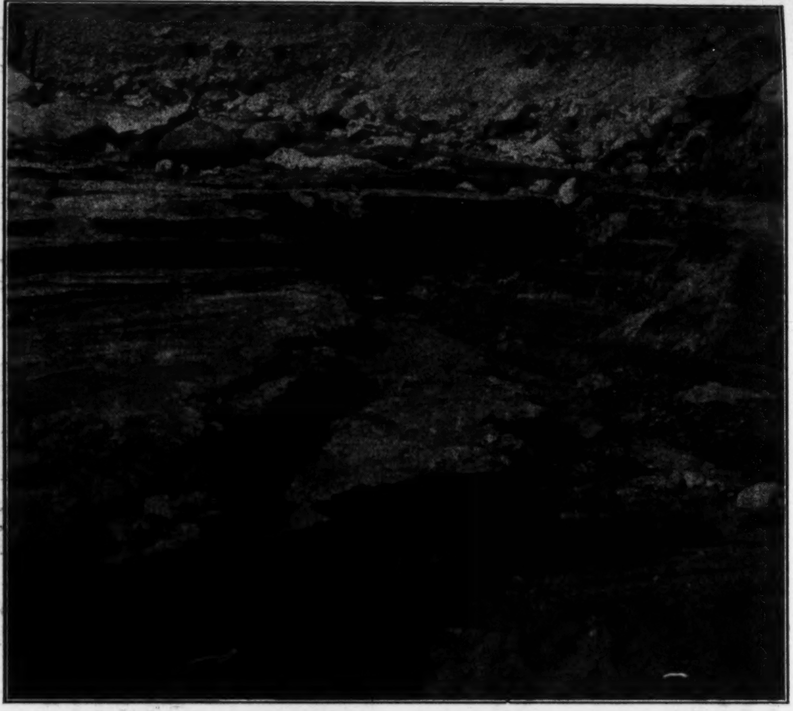

Fig. 15.—Stream-Bed Deposits in Pickens Creek, Showing How a Small Underloaded Stream Flow Had Removed Much of the Deposit

Gilbert[39] found that the addition of the finer material to the stream flow tends to facilitate the movement of the larger particles; and also that the packing together of larger and smaller grains tends to reduce the percentage of voids, which suggests that the percentage of voids, used inversely, might serve as a sort of index of mobility.

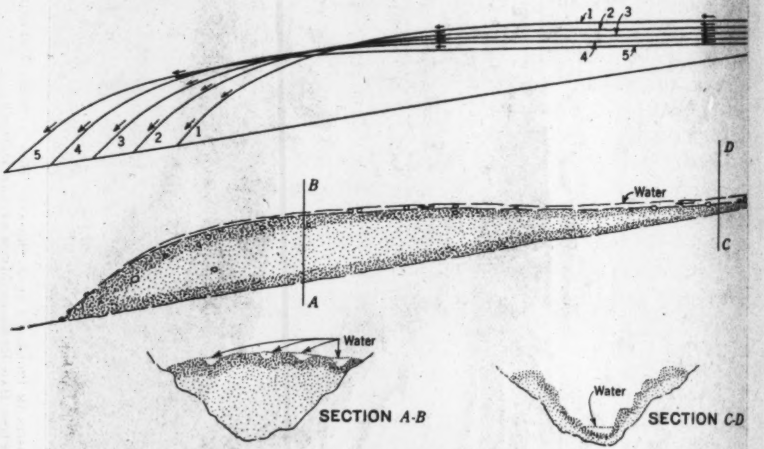

FIG. 16.—TYPICAL METHOD OF DEBRIS MOVEMENT

It is certainly much easier to conceive that a smaller force would be required to develop movement between the particles composing the debris wave, as indicated in Fig. 17, than in deposits immediately below it. It is the writers' opinion that a debris movement, such as indicated in La Cañada Valley on New Year's day, is dependent on the presence of these finer particles.

In Fig. 18 the compositions of the following seven samples are plotted:

Curve	Description
A	Deposit 3 ft deep in a house on Pickens debris cone. Boulders as large as 18 in. were common in this deposit. It showed the poorest degree of sorting and was likely deposited as a mudflow.
B	Fine-grained lens in a rudely lenticular gravel deposit at Pickens wash and Montrose Boulevard.
C	Rudely stratified deposits 4 ft deep with scattered boulders as large as 3 ft, from Halls Canyon overflow.
D	Fine-grained lens from culvert deposit at same location as the sample in curve *B*. This material probably flowed as a viscous mass.
E	From debris deposit (shown in Fig. 17) in Pickens Canyon just above Foothill Boulevard. Altitude, about 1,600 ft.
F	From debris deposit in Pickens Canyon at altitude of about 2,100 ft.
G	Typical bed load of the Santa Ana River.

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[39] "The Transportation of Debris by Running Water," by G. K. Gilbert, *Professional Paper No. 86*, U. S. Geological Survey, 1914, p. 179.

Six of these samples were taken of the debris deposits found in La Cañada Valley; the other is that of the bed load of the Santa Ana River near Prado, Calif. The data pertaining to samples *A*, *B*, *C*, and *D* were collected by W. D. Chawner.[40]

It will be noted that the bed-load sample taken on the Santa Ana River (curve *G*) contained only 3.5% of material less than 0.01 in. in diameter. This is to be expected as the finer particles are carried off as a suspended load. However, in the samples collected in La Cañada Valley area, five showed a content of 18% to 22% of particles less than 0.01 in. in diameter. Not all of the finer particles were carried off as a suspended load, probably due to the supply being considerably greater than usually encountered in this area. In fact, this large content of fine particles might indicate that there was little of the segregation usually found in flood flows, and that the finer particles have been deposited as freely as the larger. Of the nine samples collected by Mr. Eaton[41] in the vicinity of Montrose, six contained from 19% to 26% of particles less than 0.01 in. in diameter.

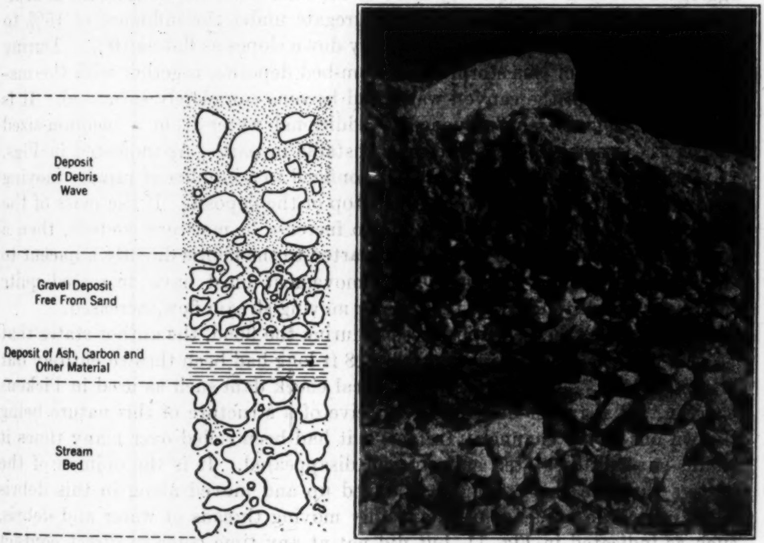
Deposit of Debris Wave
Gravel Deposit Free From Sand
Deposit of Ash, Carbon and Other Material
Stream Bed

FIG. 17.—COMPOSITION OF DEBRIS DEPOSITS IN CHANNEL OF PICKENS CREEK (ALTITUDE, ABOUT 1,590 FT)

There was considerable more variation in the composition of these samples for particles greater than 0.1 in. in diameter. The analysis of the six samples collected in La Cañada Valley showed the debris deposits to contain from 4% to 54% of particles in excess of 0.1 in. in diameter. The samples by Mr. Eaton show a similar composition except possibly with less extreme variation. Six of Mr. Eaton's nine samples showed material in excess of 1.5 in. in diameter.

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[40] "Alluvial Fan Flooding—The Montrose, California, Flood of 1934," by W. D. Chawner, *American Geographical Review*, Vol. XXV, 1935, p. 261.

[41] "Flood and Erosion Control Problems and Their Solution," by Courtlandt Eaton, *Transactions*, Am. Soc. C. E., Vol. 101 (1936), p. 1322.

Mr. Chawner,[42] in discussing his debris samples, states that complete analysis of the debris was impractical inasmuch as boulders of tremendous size were found associated with a mixture of material ranging down to fine sand. He further states that, in general, the coarseness of the material decreased from the apex of the cone and cites the presence of large boulders such as one of 32 tons at 7,000 ft from the mouth of the canyon; another of 23 tons at 7,900 ft; one of 11 tons at 8,400 ft; and one of 5.5 tons at 9,400 ft.

As already indicated, it is difficult to conceive the movement of these boulders, both large and small, out of the canyon except in the presence of debris deposits of a rather fine material. A deposit composed alone of boulders would resist any except a very large shearing force. These same boulders, however, when becoming completely surrounded by large deposits of fine material, offer considerably less resistance to the shearing forces. The presence of the finer particles, together with sufficient moisture, tends to increase the instability of the mass. In general, the analysis of this debris indicates an aggregate showing considerably more of the finer particles than found in ordinary concrete. The usual concrete aggregate under the influence of 15% to 25% of moisture is known to flow readily down slopes as flat as 10%. During the earlier phase of this storm the stream-bed deposits, together with the material eroded from the canyon walls, had become completely saturated. It is possible that under the influence of additional water from a medium-sized shower these deposits might become unstable in part. As indicated in Figs. 11 and 16, most of the movement was confined to a number of rapidly moving streams of water and debris across the top of the deposit. If the mass of the debris were to become unstable, due to increase in moisture content, then a certain amount of motion would be imparted to the debris directly adjacent to these rapidly moving streams. This movement must have decreased quite rapidly as the distance from the rapidly moving stream flow increased.

In discussing the velocities present during the flood, the author states that "a part of a check dam 4 ft by 4 ft by 18 ft long had been thrown entirely out of the channel." Fig. 10 shows a typical check dam such as used in Pickens Canyon. It is rather difficult to conceive of a structure of this nature being thrown out of the channel. In fact, if it had been rolled over many times it would have disintegrated entirely and disappeared. It is the opinion of the writers that the check dam was buoyed up and moved along in this debris wave under the influence of the rapidly moving threads of water and debris, such as indicated in Fig. 11, but did not at any time come in direct contact with these fast moving streams of water. If it had, it would have been destroyed immediately by rolling over or by impact from boulders and debris traveling at velocities such as indicated by the author. The movement of a similar, but somewhat larger check dam, is shown in the report on the "Flood in La Cañada Valley."[43]

It is not intended to minimize the high velocities deduced by the author, but merely to point out that these extremely high velocities were confined to a very small part of the large cross sections indicated in Table 1. Thus, it is the writers' opinion that the author's so-called "surges" are really "debris waves" such as indicated in Fig. 16. The author calls attention to the lack of conformity between the various sections (Table 1) when expressed as a unit of area. It is rather difficult to conceive why any conformity should be expected. These cross sections will vary considerably, even in relatively short distances and within short time intervals, depending upon the alinement, cross section, and debris movement. In general, the author's flow sections appear to be more of an index of the debris movement during the New Year's storm than an indicator of the stream flow.

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[42] "Alluvial Fan Flooding—The Montrose, California, Flood of 1934," by W. D. Chawner, *American Geographical Review*, Vol. XXV, 1935, p. 260.

[43] "Flood in La Cañada Valley, California, January 1, 1934," by Harold C. Troxell and John Q. Peterson, U. S. Geological Survey *Water Supply Paper No. 796-C*, 1937, p. 91.

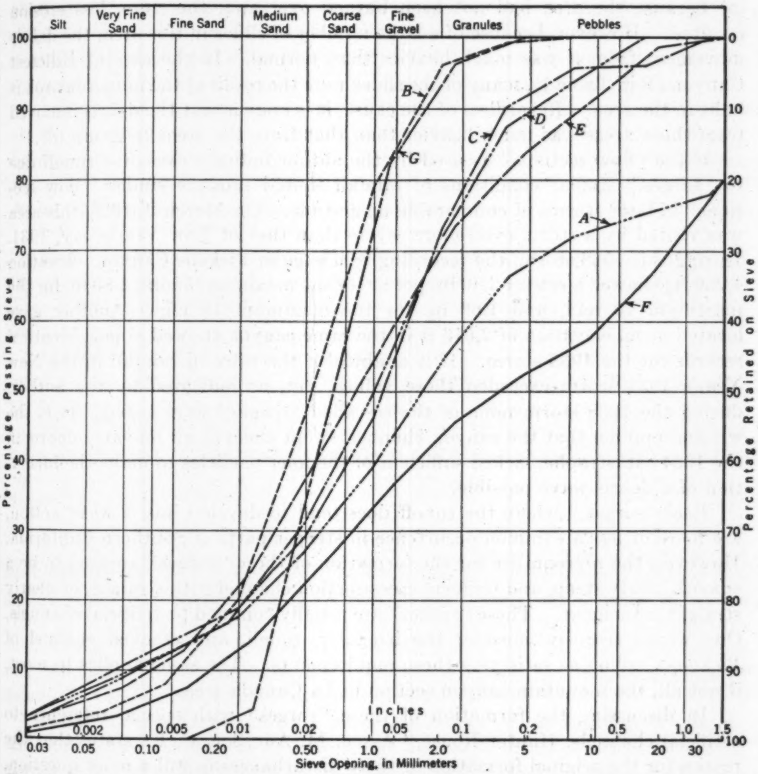

FIG. 18.—MECHANICAL ANALYSIS OF SELECTED SAMPLES FROM LA CAÑADA VALLEY AND SANTA ANA DEBRIS DEPOSITS FROM NEW YEAR'S 1934 STORM

Group (*a*) all showed large "flow sections." This is to be expected since all of the areas listed in this group are in the area burned in 1933 and the debris flow was excessive. Group (*b*) comprises the areas burned in 1927. The "flow sections" for this group are much less than those in group (*a*), mainly because the debris movement was much less. It is natural to expect less debris movement from an area burned in 1927 than from one burned in 1933, because: (1) Much of the fine material had been removed in the 6 years following the burn, and (2) there were 6 years of plant re-growth to hold the fine material back. Group (*c*) shows smaller "flow sections" than either group (*a*) or group (*b*) because the area had not been burned over and, therefore, less erosion resulted. However, because of a large number of slides in this area, the debris movement from it was much heavier than normal. In the case of Hillcrest Canyon (R in Table 1), many of the slides were the result of the numerous roads built in the area. Regardless of the cause, it is known that the debris removed from those areas was much heavier than that from the areas in group (*d*).

If the "flow sections" as used by the author indicate excessive runoff due to "surges," similar conditions of rainfall should produce similar "flow sections" in later storms of comparable magnitude. On March 2, 1938, this area was visited by a storm even more severe than that of New Year's day, 1934. During this 1938 storm, the recording rain gage in Pickens Canyon (elevation 4,250 ft) showed a rate of 2.40 in. per hr for the maximum 5 min, 1.80 in. for the maximum 10 min, and 1.60 in. for the maximum 15 min. Another gage located at an elevation of 2,616 ft in the same canyon showed almost identical records for the 1938 storm. It is doubtful if the rates of rainfall in the New Year's 1934 storm exceeded these values; yet, as indicated by the author, during the 1938 storm none of the so-called "surges" were noted. It is the writers' opinion that the canyon channel, swept clean of all movable debris in the 1934 catastrophe, lacked sufficient of the finer particles to make the formation of a debris wave possible.

Real "surges," where the runoff does tend to develop into a wave action, are more or less a common occurrence in steeper parts of Southern California. However, the prerequisite for the formation of these "surges" appears to be a smooth, fairly steep, and uniform cross-section channel with straight, or nearly straight, alinement. These "surges" are usually confined to artificial channels. One would scarcely consider the irregular, rough, and crooked channel of Pickens Canyon as satisfying these requirements. The same applies to most, if not all, the mountain canyon section in La Cañada area.

In discussing the formation of these "surges" with special reference to artificial channels, Hunter Rouse,[44] Assoc. M. Am. Soc. C. E., states that the reason for the original formation of these disturbances is still a moot question, but indicates that their formation is less likely in irregular channels.

In justifying peak discharges commensurate with the large cross-sectional areas found in Pickens Canyon, the author has implied that under certain conditions it is possible to have rates of runoff in excess of the extreme rates of rainfall. Undoubtedly, there are small drainages in which the conditions governing runoff are such that the rates of runoff may exceed the rates of rainfall supplying this runoff; but it is believed that this condition is the exception rather than the rule.

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[44] "Fluid Mechanics for Hydraulic Engineers," by Hunter Rouse, *Engineering Societies Monographs*, 1938, p. 387.

In considering the runoff in these steep mountain areas it is believed that the author does not give proper weight to the time it takes the particle of water to travel from the point where it touches ground surface as rainfall until it arrives in one of the major channels of the stream. The runoff from the major part of even these small drainages in the Sierra Madre range reaches the main channels through minute stream channels and the smaller tributaries. One is likely to forget that nearly always these steep stream channels are very rough in cross section and irregular in alinement, so that much of the advantage due to slope is counteracted by loss of energy due to turbulence. From certain work done in connection with a report on the March, 1938, flood, the writers are of the opinion that the velocities of the water in the minute channels have been generally overestimated. This work tended to indicate that the average velocity of the particles of rainfall from the time they hit the ground until they passed the gaging station was from 3 to 4 ft per sec.

Inasmuch as these velocities are very much slower than those present in the main stream channel in each of these areas, it is apparent that travel down the mountain slopes must be considerably slower than these average velocities. Thus it seems likely that the author should have given more consideration to the time it takes the particles of runoff to reach the main channels.

As indicated by this discussion, large cross sections are not always a true index of the peak discharge. From time to time hydrological literature cites very large rates of runoff, ranging (as stated by the author) from 1,000 to 4,170 cu ft per sec per sq mile. If these values are the result of determinations made subsequent to the peak flows where the cross-sectional areas may have been subject to considerable debris movement, they should be used with a degree of caution.

KARL J. BERMEL,[45] ESQ. (by letter).—The rates of runoff that would be necessary to obtain the flow sections from zones of excess runoff are, as the author states (see text supporting Table 2), out of proportion to the flow that could be sustained by the recorded rainfall. The supposition that a maximum momentary flow section greater than could be sustained by the entire rainfall can be attributed for the most part to the formation of flood surges brings to mind other factors that should be considered.

In a problem of this nature the two factors—rainfall, or estimate of rainfall, and runoff, or estimate of runoff—influence the rainfall-runoff relation. A variation in the measurement or estimate of either quantity from that which actually occurred would lead to erroneous results. Since the paper deals primarily with runoff, this factor will be considered first.

Attention should be drawn to the fact that runoff containing debris from a given area cannot be expressed as a percentage of rainfall on that area unless the quantity of transported material is negligible. If it is assumed that the entire rainfall becomes runoff and that there is no transported material in the flow, the runoff would amount to 100% of the rainfall; but if this runoff is increased in volume by transported material, the combined total, as indicated by a flow section, could not be accounted for by rainfall alone. This is particularly true when the runoff from zones of excess runoff contains as much as 50% solids by volume (heading "Debris Content").

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[45] California Forest and Range Experiment Station, U. S. Forest Service, Berkeley, Calif.

From the author's statement, preceding Equation (1), that "No attempt has been made to compute the peak flows" (from zones of excess runoff), it seems apparent that the statements following Table 2—"or 15 times larger than the average flows in the normal zone"; "or 23 times larger than average flows in the normal zone"; and "or 57 times larger than the average flows in the normal zone"—should read "larger than the flow sections in the normal zone."

Reference is made to the statement following Table 2 and the list of average measurements: "A flow of 4,000 cu ft per sec per sq mile requires a runoff at the rate of more than 6 in. in depth each hour, and in these streams would have a section of approximately 130 sq ft for each square mile." Although this statement is true for clear water, it is not a true index of the flows that would be necessary to create their respective flow sections in the zones of excess runoff.

In the foregoing example a value of approximately 31 ft per sec is used as the average velocity; but from the evidence presented in the paper this value is more nearly that of the maximum surface velocity. The average velocity would be less than the surface velocity, especially if the surface velocity is that of a surge, for as Mr. Lynch states (see heading "General"): "With a runoff that is growing greater there is a tendency for the deeper, faster moving water from upstream to overtake the water in front of it."

This is also demonstrated by the following formula[46] in which the speed of the front of the surge is

$$\omega = V + \sqrt{g\left(\frac{A}{B} + \frac{3}{2}Z\right)} \qquad (3)$$

In Equation (3) V = the mean velocity of water in the channel before the surge; A = the original channel cross section; B = the original channel width; g = the acceleration of gravity; and Z = height of the front of the surge above the original water surface. With a reduced average velocity, a flow of 4,000 cu ft per sec per sq mile would require a flow section larger than 130 sq ft per sq mile. Furthermore, when this clear-water flow was bulked by debris, as it would be if it were representative of flows in zones of excess runoff, the flow section would be increased materially, not only by the bulking, but also by a lowered average velocity occasioned by this debris.

As a further consideration of runoff exceeding the flow that could be sustained by the rainfall, variations between measured rainfall and rainfall that is actually intercepted by a watershed may also lead to erroneous rates of runoff expressed as percentage of rainfall.

The major factors that contribute to the error of using rainfall, as measured by a standard U. S. Weather Bureau rain gage, as a measure of watershed interception, excluding the factors of number and location of gages, are the inclination of rain, the gradient of the watershed slopes, and the aspect of these slopes with respect to the rain inclination. The following theoretical considerations are presented to illustrate the fallacy of using rainfall, as recorded by a standard rain gage, as a criterion of rainfall intercepted by a watershed.

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[46] "Hydraulic Structures," by Armin Schoklitsch, Vol. I, p. 123; translated by Samuel Shulits, A.S.M.E., 1937.

In Figs. 19 and 20 the following quantities are used: Area of topography, 1 sq ft (horizontal projection); area of rain gage, 1 sq ft; rainfall, 1 in. per hr for a duration of 1 hr, represented by a uniform spacing of 10 lines per ft on a line at right angles to the direction of rainfall; and inclination of rain and slope of topography are assumed as 45° for purposes of illustration.

In Fig. 19(*a*) the topography is at any gradient and the rain falls perpendicularly. In Fig. 20 the topography is level and the rain falls at any inclination. These are the only conditions under which recorded rainfall is a measure of surface interception. It is apparent in Fig. 19(*a*) that the rainfall intercepted by either the level or the sloping topography or the rain gage is the same—that is, a depth of 1 in., or, on the basis of a horizontal area of 1 sq ft, 144 cu in. The depth on the slope area is obtained as follows: $\frac{b}{c} = \cos\alpha$; and $c = \frac{b}{\cos\alpha} = \frac{1}{0.7071} = 1.414$ ft. The slope area equals $1.414 \times 144 = 203.62$ sq in.; the ratio $\frac{\text{Volume of rain}}{\text{Slope area}}$ = depth per area in inches = $\frac{144}{203.62} = 0.7071$ in.

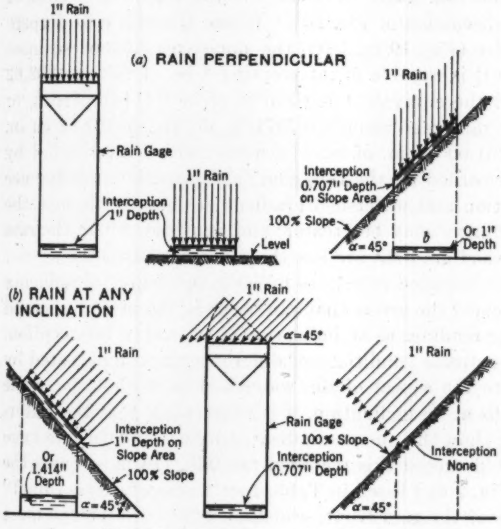
(a) RAIN PERPENDICULAR
1" Rain
Rain Gage
Interception 1" Depth
1" Rain
Level
1" Rain
Interception 0.707" Depth on Slope Area
c
100% Slope
α=45°
b
Or 1" Depth
(b) RAIN AT ANY INCLINATION
1" Rain
Interception 1" Depth on Slope Area
Or 1.414" Depth
100% Slope
α=45°
1" Rain
α=45°
Rain Gage
100% Slope
Interception 0.707" Depth
1" Rain
Interception None
α=45°

FIG. 19.—TOPOGRAPHY AT ANY SLOPE

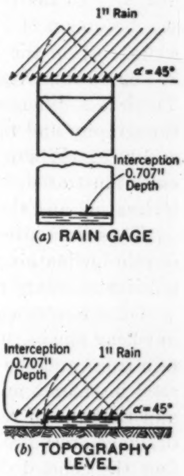
1" Rain
α=45°
Interception 0.707" Depth
(a) RAIN GAGE
Interception 0.707" Depth
1" Rain
α=45°
(b) TOPOGRAPHY LEVEL

FIG. 20.—TOPOGRAPHY LEVEL; RAIN AT ANY INCLINATION

In Fig. 19(*b*) it is apparent that the rain gage and the level topography intercept equal quantities of rainfall, not taking into consideration differences due to such factors as wind currents and splash. The rainfall that is intercepted, however, depends upon the area capable of interception, which is the horizontal area projected at right angles to the direction of rainfall. The projection of any plane area upon another plane is the product of that area and the cosine of the angles between the planes. Since the horizontal area is 1 sq ft and the cosine of 45° = 0.7071, the projected area equals 1 times 0.7071 = 0.7071 sq ft or 0.7071 times 144 = 101.82 sq in. Since 1-in. depth of rainfall is intercepted on this projected area, the volume of rain intercepted equals 101.82 times 1 = 101.82 cu in.

Fig. 19(*b*), in which the topography is at any gradient (except level) and the rain falls at any inclination, illustrates the error that can be introduced by the use of recorded rainfall as a measure of surface interception. Where the rain falls parallel to the surface of the topography (Fig. 19(*b*), right) no interception can occur; the rain gage, however, records 0.7071-in. depth or 101.82 cu in. of rain (see discussion of Fig. 20). Where the rain falls perpendicularly to the topography (Fig. 19(*b*), left), the slope area (203.62 sq in.—see discussion of Fig. 19(*a*)) is capable of intercepting 1 in. of rain, or 203.62 cu in., or, on the basis of the equivalent horizontal area, 1.414-in. depth, or 203.62 cu in. Again, the rain gage records 0.7071-in. depth, or 101.82 cu in. There is a difference of 101.60 cu in. of rain between rainfall intercepted by topography and rainfall recorded by the rain gage, an approximate difference of 100%. Where inclination and watershed gradient are greater than in the cases illustrated, the differences may be greater; and, likewise, where the rain inclination and the watershed gradient are less the differences are less.

These instances are not intended merely to indicate the degree of influence of rain inclination; they reveal the errors that can occur in the present method of using standard rain-gage readings as an indication of watershed interception.

On a watershed these extreme conditions of difference are compensated by opposing slopes, but where the aspect of the watershed as a whole faces the rain front and the rain falls at an inclination, it is conceivable that the watershed intercepts more rain than the amount indicated by a standard rain gage and that runoff rates would exceed this recorded rainfall. In most cases the drainage areas shown in Fig. 2 and listed in Table 1 as "zones of excess runoff" face the general direction of the rain front, and they tend to intercept more rain than the quantity recorded. On the other hand, the aspect and lower gradient of Sycamore Canyon, described under "Steepness of Canyons," might have been favorable to actual interception nearly equal to observed rainfall and consequently to more normal runoff.

In predicting runoff rates, or in assigning various coefficients to runoff, it is necessary first to be able to ascertain the amount of rainfall intercepted by a watershed. Rainfall records at best provide only an indication of the actual rainfall for an area. However, rainfall recorded by a slope rain gage would be a better indicator of actual watershed interception, especially in steep watersheds. For the slope gage to intercept rainfall representative of the area it samples, the plane of each gage must be placed in the average plane of the topography it is to represent. The factors of slope, aspect, and rain inclination need not then be considered. However, the number of gages, the spacing, and the other considerations that arise in initiating installations for proper sampling of an area will present even more difficulties than are now encountered in the installations of standard rain gages.

R. W. DAVENPORT,[47] M. AM. SOC. C. E. (by letter).—The engineers of the U. S. Geological Survey, who are engaged in determining the flow of surface streams, and in correlating stream flow with rainfall and other influential factors, will find this paper especially interesting.

The determination of the flow of surface streams relates, of course, to the entire range of flow, from low water to floods. The objectives of the U. S. Geological Survey are to collect and publish, in such detail as is practicable, the information that will be needed by users of the records. An increased interest in flood data, coincident with a period of greater prevalence of outstanding floods, has been met by a pronounced effort to satisfy the needs of all concerned as adequately as possible. Consequently, greater emphasis is being given to supplying flood data in the annual reports on surface-water supply, and several special flood reports and compilations have been published. Valuable contributions to special reports of that kind have been made by many public and private agencies.

The general objective of the special reports is to furnish, so far as practicable, records of stage and discharge at regular gaging stations throughout floods of major consequence at sufficiently close intervals to permit a satisfactory reproduction of the actual hydrograph, and to furnish peak discharges of the flood at miscellaneous places of interest where continuous observations are not available. Determination of dependable peak discharges is obviously a problem of major importance.

Generally, in all except the largest rivers, flood peaks have the characteristic of being comparatively brief and transitory and of occurring in many streams almost simultaneously. Because of the usual interference with, or paralysis of, transportation facilities by abnormally high floods, the widely separated locations of gaging stations, and limited engineering personnel, it is rarely possible, particularly on the smaller streams, to have an engineer on the ground to observe the passing of the peak. Because of the wild and destructive behavior of floods, as well as the great quantity of debris often carried along by the water, it is commonly quite impossible to obtain a direct measurement of peak discharge. This situation requires that efforts be made to determine peak discharge by obtaining as much physical evidence reflecting the magnitude of flow as possible for analysis by suitable hydraulic formulas, such as the Manning formula (using slope and cross-sectional area), the formula for flow over weirs, the formula for flow through contracted openings, and others of less general utility. The factors involved are too often far outside the range of guiding experience with the formula used. In most instances the exercise of a large degree of mature judgment, which can be gained only by wide experience, is required.

The complex problems related to determination of peak discharges is particularly challenging in floods of the so-called "cloudburst" type. Such floods occur with varying degrees of frequency in different sections of the United States. They are associated mostly with comparatively small drainage basins. Where they have occurred on streams draining larger basins, it appears that the excessive rainfall which caused them was largely limited to relatively small parts of the basins. Cloudburst floods in the arid regions are commonly associated with the movement of large quantities of debris of various kinds. In humid regions the movement of debris, although characteristic, tends to be a less prominent feature.

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[47] Hydr. Engr., U. S. Geological Survey, Washington, D. C.

If a surge-like flood of water, unmixed with debris, were passing down a channel that previously was essentially empty, the magnitude of the surge would tend to diminish progressively because of steady depletion by channel storage. The surge could be maintained at its original magnitude, other conditions remaining favorable, only if tributary inflow were properly timed and equivalent to the depletion in channel storage. Thus the conditions requisite to the persistence of a surge of water free of debris in a natural channel seemingly include a peculiar synchronization or combination of high rates of inflow from tributary drainage. A similar coincidence of high inflow is even more essential to the original generation of a surge.

Studies of the relation of rainfall and runoff have indicated that appreciable quantities of rain are accounted for as surface detention—water which wets the ground surface and fills the infinite number of more or less minute depressions, as distinguished from infiltration, or absorption, and runoff. If a torrential rainfall should happen to come at a time when surface detention was relatively large, due to favorable antecedent conditions such as the accumulation of vegetation and litter, it is conceivable that a substantial part of such surface detention might quite abruptly become available as runoff through breakdown or failure of the obstructions storing the water. The runoff from the torrential rainfall would therefore be augmented perhaps by a substantial percentage through the sudden release of surface detention, or the failure of innumerable miniature reservoirs. The drainage area over which such phenomena would be likely to occur simultaneously would seemingly be relatively small. The writer conceives that an event of this kind may be one of the factors in generating surges in cloudburst floods.

If a flood of water unmixed with debris were collecting in a channel system and passing down the main channel as rapidly increasing discharge, and if in the forepart of the flood a large quantity of mud, gravel, boulders, and all kinds of trash were introduced, the effect on the flow would be pronounced. With debris composed of finer grades of mud and gravel the moving mass would in some measure be comparable to a liquid, but a liquid that behaves quite differently from clear water. With coarser debris consisting of boulders, logs, and trash the upset in hydraulic behavior would also be striking. The velocity would be slowed down by absorption of energy needed to carry the debris down slopes flatter than the dry angle of repose or by reduced liquidity or greater viscosity and consequent internal friction. It would tend to create a relatively slow moving mass which would be overtaken and enlarged by the more rapidly moving water and debris that is following. In a sense it would behave like a dam.

Obviously, the effect of any other factors tending to produce a surge of flow in the channel, such as torrential rainfall, favorable concentration of runoff, and other influences, would be increased by a substantial increment of debris. Probably this kind of phenomenon is largely responsible for the frequently reported wall-like aspect of the front of the wave and other surge-like features common to cloudburst floods.

The actual discharge represented by such a moving dam with its following pond of water and debris would seem to depend not only on the rate of movement of the dam and the rate of flow of the oncoming water, but also upon what portion of such oncoming water goes into temporary channel storage behind the moving dam. Rupture of the dam and abrupt release of the retarded water would doubtless result in very high rates of discharge until impeded again by the accumulation of more debris and further reduction by channel storage. The conditions which arise from the breakdown of one surge of this kind may react to form another. The widening or narrowing of a channel may also have an important influence on the manner of progress of a surge. Unfortunately, the lack of even the most fragmentary field data on these phenomena prohibits the crudest form of analysis. The author's finding that floods of the "cloudburst" type yield momentary runoff peaks entirely out of proportion to the rates of rainfall is apparently based on information other than the direct comparison of observed rates of rainfall and runoff. In surge-flow peaks the effects of channel phenomena may obscure any satisfactory quantitative relation with rainfall rates.

The cross-sectional areas listed in Table 1 may be less indicative of flowing water than of a more viscous, slower flowing liquid heavily loaded with debris. It is obvious that the movement of large quantities of debris may seriously complicate the determination of rates of flow by hydraulic formulas or any other means.

The following quotation from a report[48] of the U. S. Geological Survey on the New York State flood of July, 1935, discusses some of the difficulties encountered in making flood determinations, and issues cautions as to their use:

> "The application of formulas and coefficients used in the computation of the flood flows was made with a full appreciation of the limitations of scientific knowledge of the behavior of streams under unusually extreme conditions such as those of the July flood, and is believed to be consistent with good engineering judgment. Many of the streams undoubtedly carried enormous quantities of debris. The effect of this debris upon the applicability of the laws and formulas generally accepted as governing the flow of water is problematic. The same statement applies to the effects of sediment, entrained air, turbulence, excessive slopes and velocities, and other factors, which occurred in a degree far outside the field of ordinary experiment and experience. * * * It is believed that, through the reaches selected for the determination of flood discharge, the debris moved downstream in such a manner as to cause very little if any reduction in apparent area or water capacity of the channels. For the purposes of this report it has been assumed that the water surface of the streams was represented by the high-water marks indicated on the banks, that the channels as surveyed had remained substantially unchanged throughout the flood, and that the flow conformed to the laws of the flow of water expressed by the formulas selected for the determination of the particular flood discharge. The results thus obtained are believed to be in the most useful form and of such value in the planning of flood-protection measures as to warrant their publication as the most reliable data

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[48] "The New York State Flood of July, 1935," by Hollister Johnson, Assoc. M. Am. Soc. C. E., U. S. Geological Survey, *Water Supply Paper No. 773-E*, p. 251.

that can be supplied. However, any user of the data is cautioned to keep in mind the method of derivation and to make such allowance therefor as may seem appropriate."

With all the attendant difficulties and resulting uncertainties that may exist in the determination of discharge by direct measurement, or indirect methods, it should be recognized that certain inaccuracies are inherent and recognizable in the determination of supply by rainfall observations. Unfortunately, the measurement of amounts, duration, and intensity of precipitation is fraught with some difficulty, particularly on an areal basis such as that usually associated with cloudburst floods.

Approaches to the problem on the broad basis of available knowledge of rainfall rates, infiltration rates, etc., may contribute materially to an understanding of the flow characteristics of transient flood peaks. Investigations of the volume of water required to transport measurable deposits of debris may be illuminating.

The author has presented interesting and valuable information concerning floods of the cloudburst type. From the standpoint of a satisfactory and complete understanding of the associated phenomena, the exploration of the field has only begun. With the accumulation of observations and experience, both in the field under natural conditions and in the laboratory, the prospects for better understanding are promising.

Walter J. Wood,[49] and Maxwell F. Burke,[50] Assoc. Members, Am. Soc. C. E. (by letter).—An interesting treatment of a subject not new, but still controversial in many respects, is presented in the paper. Debris flows are of particular interest to engineers in Los Angeles County because of potential hazards to life, property damage, and silting of flood control and conservation reservoirs and debris basins.

The use, as a unit for comparison, of the "cross-sectional area per square mile of catchment area" is well chosen. Comparison by the usual method of discharge in cubic feet per second, or in second feet per square mile, is inadvisable in the case of this flood due to the almost complete lack of factual data on velocities and other hydraulic properties taken during the peak of the flood. The absence of gaging stations, except in Haines Canyon, and the inability of hydrographers to travel over flooded roads precluded obtaining flow data. Numerous lay estimates were furnished by various observers but they varied widely. The many flow-section areas obtained by the author are probably reliable for comparison of surges but certainly would not be reliable for use in the computation of discharge.

Although the rainfall conditions described by the author may produce the cross-sectional areas of flow measured by him, wave formation may occur even if it is unaccompanied by a sudden rainfall increase. The formation of waves, termed "traveling waves" or "slug flow," can increase the cross-sectional flow area to many times that occupied by the average flow. This is demonstrated by the observations reported by W. H. Holmes,[51] Assoc. M. Am. Soc. C. E., in 1936. The flow observed on January 5, 1935, at Mountain Street in the Verdugo Channel, was recorded by a water stage recorder about a mile and one half upstream. The average flow at the gaging station was about 1,100 cu ft per sec. This checks closely the flow computed from Mr. Holmes' estimates of hydrograph shape and his measurements of wave frequency; yet the maximum cross section occupied by the wave peak was 43 × 8 = 344 sq ft, compared with a flow cross section at the gaging station of 45 sq ft, a ratio of 7⅔ : 1. The cross section, slope, and construction of the channels at the two locations were practically identical. Assuming the velocity of water in the wave front to be equal to the observed velocity of the wave, the peak flow amounted to 43 × 8 × 23.5 = 8,100 cu ft per sec; yet the peak flow at the gaging station above was not more than 1,500 cu ft per sec at the maximum. This represents an increase of 440% in peak flow due to channel hydraulics alone, without any benefit from rainfall variation, burned watershed, small catchment area, fan shape of tributary collectors, steep slopes, or any of the other factors assumed by the author to be contributing to flood wave production.

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[49] Civ. Engr., Grade III, Los Angeles County Flood Control Dist., Los Angeles, Calif.

[50] Hydr. Engr., Los Angeles County Flood Control Dist., Los Angeles, Calif.

[51] "Traveling Waves in Steep Channels," by W. H. Holmes, *Civil Engineering*, July, 1936, p. 467.

Similar observations made in concrete channels in the Montrose area during the flood of March 2, 1938, indicate that, for an average flow called 100%, the variation of flow rates was from 10% to 425% of this average due to the formation of these "traveling waves." These observations were made in the channels below debris basins, where variations in inflow rate had been smoothed out to very minor fluctuation in outflow rate. The water in these waves was not carrying debris other than silt in suspension.

During the storms of December 14, 1934, and January 5, 1935, observers were stationed in Pickens Canyon and Hall-Beckley Canyon to make observations on flows. Traveling waves developed in both these storms with waves occurring at intervals of 11 to 12 sec (between 7:58 a.m. and 8:36 a.m., in Fig. 21). Observations were made on random waves and a gage-height record was kept so that the average flow and the peak flow rates could be approximated. The hydrograph obtained from observations in Hall-Beckley Canyon

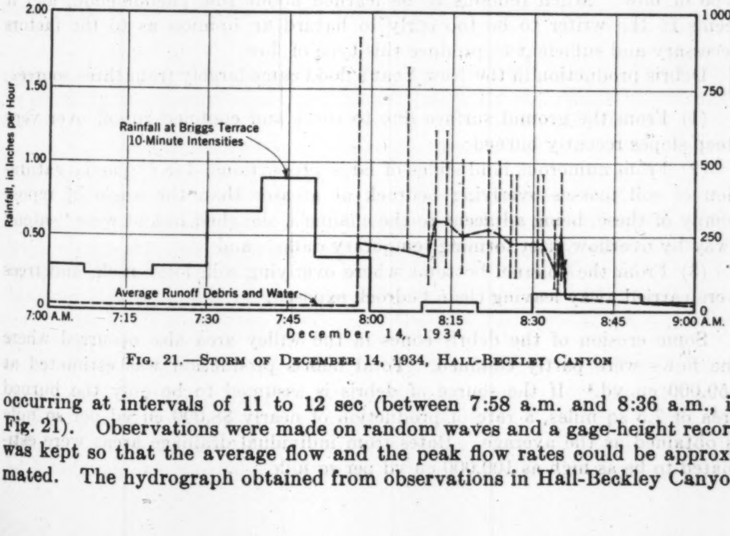

FIG. 21.—STORM OF DECEMBER 14, 1934, HALL-BECKLEY CANYON

during the storm of December 14, 1934, is shown in Fig. 21 as a sample of this type of information. The vertical broken lines in Fig. 21 indicate random waves that were timed through the gaging reach. During this storm, in Hall-Beckley Canyon (drainage area, 0.77 sq mile burned) the maximum observed ratio of surge flow to mean flow was 925 cu ft per sec to 230 cu ft per sec, or 4 : 1. The maximum observed ratio of peak flow to minimum flow was 610 cu ft per sec to 150 cu ft per sec, again 4 : 1. At no time during the storm was the rainfall rate greater than 0.90 in. per hr for a 10-min period. During the January storm the maximum observed ratio of surge flow to mean flow was 2,110 cu ft per sec to 420 cu ft per sec, or 5 : 1. The maximum observed ratio of peak flow to minimum flow was 2,110 cu ft per sec to 140 cu ft per sec, or 15 : 1. At no time during the storm was the rainfall intensity greater than 0.95 in. per hr for a 10-min period.

For the January storm in Pickens Canyon, the maximum observed ratio of surge flow to mean flow was 1,050 cu ft per sec to 680 cu ft per sec, or 1.5 : 1. The maximum observed ratio of peak flow to mean flow was about 1,090 cu ft per sec to 600 cu ft per sec, or 1.8 : 1. The maximum 10-min rainfall intensity during the storm was 0.95 in. per hr.

This type of surge flow, or "traveling wave" flow, has been observed in Los Angeles County under practically every condition—with and without included debris, in natural channels and in artificial channels where normally Kutter's "*n*" value would vary from 0.014 to 0.035 or more, from mountainous drainage areas and from purely urban areas, for rectangular channels and for trapezoidal channels, for relatively large flows (4,500 cu ft per sec or more) and for very small flows—in fact, the only factor consistently present in slug flow phenomena appears to be that the velocity of flow must be supercritical. On the other hand, many flows at supercritical velocities do not produce this type of flow. Much remains to be learned about this phenomenon, and it seems to the writer to be too early to hazard an opinion as to the factors necessary and sufficient to produce this type of flow.

Debris production in the New Year's flood came largely from three sources:

(1) From the ground surface due to sheet and confined runoff over very steep slopes recently burned;

(2) From numerous land slides of large proportions, due to partial saturation of soil masses overlying bedrock at greater than the angle of repose (many of these, being adjacent to the channels, sloughed in and were "eaten" away by overflow after forming temporary dams); and

(3) From the channel bottoms where overlying soil, loose rock, and trees were carried away leaving clean bedrock exposed.

Some erosion of the debris cones in the valley area also occurred where the flows were partly confined. Total debris production was estimated at 659,000 cu yd.[4] If the source of debris is assumed to be only the burned area of 7.5 sq miles, a rate of production of nearly 88,000 cu yd per sq mile is obtained as the average. Rates from individual drainage areas were estimated to be as high as 100,000 cu yd per sq mile.

Subsequent to the flood of January 1, 1934, seven debris basins were constructed in the area in addition to that at Haines Canyon and the Verdugo Debris Basin which was completed after the flood. The value of these basins was evident after the flood of March 2, 1938. These basins furnished the opportunity to obtain very good quantitative data on debris production from the same canyons discussed by the author. A close comparison cannot be made between debris production in the two floods, because the estimates for the 1934 flood were necessarily somewhat inaccurate due to the large proportion of debris deposited on streets and property, and also because an unknown quantity came from erosion on the valley floor below the canyon mouths. However, even a rough comparison is interesting.

Table 9 shows the results obtained from the March 2, 1938, flood, which produced 541,500 cu yd of debris. This came from the same drainage area that produced approximately 659,000 cu yd on January 1, 1934. The difference is probably due in part to the partial restoration by regrowth of cover on the area in the 4-yr intervening period, in part to lack of debris in the channel which had already been cleaned out four years earlier, and to differences in rainfall rates.

TABLE 9.—ANALYSIS OF DEBRIS DEPOSITED IN "MONTROSE AREA" DEBRIS BASINS FROM MARCH 2, 1938, FLOOD*

Debris basin	Drainage area, in square miles	Year watershed burned	Percentage of area burned since 1927	Debris deposited in basin, in cubic yards	Debris inflow to basin, in cubic yards per square mile	Cost of debris inflow, in dollars‡
Haines	1.53	1933	31	51,500	34,000	15,450
Dunsmuir	0.84	1933	100	58,500	70,000	17,550
Shields	0.27	1933	85	32,500	120,000	9,750
Eagle-Goss	0.61	1933	79	40,700	67,000	12,210
Hall-Beckley	0.84	1933	84	93,600	111,000	28,080
Pickens	1.84	1933	92	150,200	82,000	45,060
Snover	0.23	1933	100	16,900	73,000	5,070
Hay	0.20	1933	80	12,600	63,000	3,780
Verdugo	15.24†	1927 and 1933	49	85,000	5,600	25,500
Total				541,500		162,470

* Based on records of Los Angeles County Flood Control District which operates and maintains these basins. † Partly controlled by other basins. ‡ Based on desilting cost of 30 cents per cubic yard.

The comparison of rainfall for the two storms, as shown in Table 10, may be pertinent. The January 1, 1934, storm lasted approximately 56 hr with 90% of the rain falling in the last 24 hr. The March 2, 1938, storm was broken up into two parts, the first lasting from February 27 to March 1, and the second and more severe part starting on March 2 and in general ending on March 3. The average duration of the first part was 48 hr, and of the last, 32 hr.

Table 10 shows the storm totals of March 2, 1938, to exceed those of the New Year's storm in this region; 52% of the total storm rainfall at the stations listed occurred on March 2. Maximum hourly rainfall amounts varied from 0.63 in. to 1.31 in., this latter amount being nearly the same as the maximum of 1.33 in. in one hour which obtained at the Flintridge Fire Station in 1934.

In general it may be said that, whereas the New Year's storm culminated in an abrupt increase in rainfall intensity producing high peak flows for a short time, which rapidly dropped to negligible flows, the March 2 storm rainfall held up at moderate intensities for a much longer period, producing greater flow volumes but no peak flows of extremely large proportions in the area under discussion.

TABLE 10.—COMPARISON OF STORM RAINFALL, 1938*

No.	Station	Feb. 27	Feb. 28	Mar. 1	Sub-total	Mar. 2	Mar. 3	Mar. 4	Sub-total	Grand total	Storm total
251	La Crescenta	1.21	3.35	3.08	7.64	10.27	0.89	....	11.16	18.80	13.50
280	Flintridge Fire Station	1.46	2.71	2.35	6.52	7.77	0.64	....	8.41	14.93	14.92
364	Lower Haines	1.27	2.70	2.36	6.33	7.80	0.85	0.02	8.67	15.00	10.52
360	Haines Debris Basin	1.20	2.58	2.22	6.00	7.47	0.75	....	8.22	14.22	9.37
367	Upper Haines	1.57	3.31	2.95	7.83	10.39	1.16	0.02	11.57	19.40	12.00
373	Briggs Terrace	1.18	3.36	3.15	7.69	9.98	0.86	....	10.84	18.53	12.95
378	La Cañada	1.55	3.20	2.65	7.40	8.99	1.09	....	10.08	17.48	....†
401	Verdugo Mountain	1.42	2.36	2.66	6.44	7.85	0.86	0.02	8.73	15.17	....†
418	Pickens Plots	1.43	3.60	3.14	8.17	8.12	1.06	0.01	9.19	17.36	....†
508	Arroyo Seco Ranger Station	1.26	2.62	2.62	6.50	8.10	1.00	....	9.10	15.60	11.33
577	Los Angeles, U. S. Weather Bureau Station	1.47	2.85	0.08	4.40	6.17	0.49	....	6.66	11.06	8.27
122	Pickens Debris Basin	1.22	2.49	2.64	6.35	8.40	0.57	....	8.99	15.34	....†
647	Tujunga	1.01	2.12	2.07	5.20	6.02	1.88	....	7.90	13.10	11.76
556	La Crescenta	1.18	3.76	3.05	7.99	9.42	0.94	....	10.36	18.35	13.28

* February 27–March 4, 1938, data extracted from Report on Flood of March 2, 1938, by Los Angeles County Flood Control District. Values are not maximum 24-hr rainfall. † Not available—stations started after January 1, 1934.

The most important comparison of the two floods, and one which shows the actual value of the debris basins, lies in the relative amount of damage caused by water and debris with and without the basins. The flood of January 1, 1934, caused the loss of 34 lives and wrecked 483 houses. It deposited approximately 535,000 cu yd of debris on streets and property. The total damage was estimated at $5,000,000. The debris basins impounded 541,500 cu yd of debris in the 1938 flood which is practically the same volume as was deposited on streets and property in 1934. Almost no damage was done in this region in 1938 and the total cost of desilting the basins was only $162,470, or one thirtieth of the amount of damage in 1934. The right-of-way and construction cost of the nine basins listed, and their concrete outflow channels, did not exceed approximately $3,500,000, which is considerably less than the estimated amount of damage in the one flood of January 1, 1934.

The Los Angeles County Flood Control District has in operation at the present time (1940) a total of sixteen debris basins in the Montrose and other areas. One more basin is under construction and still another will be started soon. They have proved their value in areas where burns have occurred and where debris flows may occur above highly developed residential areas.

FRANKLIN THOMAS,[52] M. AM. SOC. C. E. (by letter).—The combination of circumstances which produced the "New Year's Day Flood" of 1934 is of such rare occurrence that a repetition in the same general region is almost outside the range of reasonable probability. However, in presenting for reference through this paper his description of the phenomena which developed, and his analysis of them, Mr. Lynch has expanded the horizon within which engineers should direct their studies when considering the nature and scope of protective works to care for runoff from watersheds of steep gradient.

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[52] Prof., Civ. Eng., California Inst. of Technology, Pasadena, Calif.

In a great majority of the years, the initial rains of the season on the mountain watershed concerned in this paper occur in October or November in sufficient amount to dispel the summer-long fire hazard for the brush covering of the mountain sides. During the autumn of 1933, the rainfall as recorded by the Pasadena (Calif.) station, which is representative for the watershed, was 0.48 in. in October, 0.20 in. in November, and 10.34 in. in December. The devastating fire which was a major contributing cause of the flood denuded the mountain sides over a 5-mile front on November 21, 1933, an exceptionally late date for such an extensive fire.

It can be stated as almost axiomatic that no floods will develop from the mountain watersheds of Southern California until a rainfall of heavy intensity, extending over several hours, occurs subsequent to an aggregate rainfall exceeding 10 in. within the previous two weeks which has saturated the watershed. Such conditions seem to occur about once in a decade in the region under discussion, but only twice did they occur during the 60-yr period of record earlier in the season than the latter part of January. Those instances were in December, 1889, and December, 1921. Although the December storm which caused the local flood described by the author did provide the basic precipitation of 10 in., there was lacking the supplementary rainfall to produce a flood of serious magnitude from unburned areas.

The freshly burned area, lacking the protective cover, yielded a much earlier, as well as much greater, runoff than a corresponding surface in natural condition produced. The storm was one of sustained precipitation for at least fifteen hours before the burned watershed surfaces melted away into the canyons and succeeding washes. During the afternoon of December 31, recording gages in Pasadena showed a uniform unvarying rainfall of 1 in. per hr for three successive hours, with no cessation of earlier and later intensities which were almost as heavy.

The conditions of this storm were markedly different from those which caused the Utah floods of 1930 following abrupt precipitation of high intensities. The warning to engineers which this New Year's storm presents with its huge mass of water and debris is significant for its quantitative excesses. It compels a new approach to consideration of drainage openings, channel capacities, and probable life of reservoir space.

It is unfortunate that so much determination of what actually took place must be dependent upon hypothesis and conjecture. The author's support for his explanation of the surge effects in excess of the sustained runoff is well presented.

Other aspects of the situation, either involving debris in rapid movements along the channel, or debris which is being gradually shifted about in the channel, offer alternative bases for interpreting the phenomena described and point out new directions for observation and study.

Henry B. Lynch,[53] M. Am. Soc. C. E. (by letter).—The purpose of this paper was to call attention to, and present data relating to, the excessive cross sections characteristic of the so-called "cloudburst" type of floods, as well as to give a satisfactory explanation of them. Such floods have not received the consideration due them from engineers. This condition is due equally to lack of data and lack of discussion. The areas generating the floods have almost always been uninhabited and devoid of rain gages. As a result, it has been simple to assume that floods were caused by extremely heavy rates of mountain rainfall.

The Southern California flood of January 1, 1934, was unique among floods of this type because of the mass of available, pertinent data. In the Sierra Madre, within the area generating the floods, were five rain gages, two of which were automatic. Eight other gages were located in the thickly populated section within one mile of this area. These gages gave consistent rainfall data. Many people living within, or immediately adjacent to, the mountains were interviewed. All evidence showed that rainfall rates were only moderately high. The excessive cross sections were measured on all sides of the gages and inhabited locations. As a result of the information available, the usual explanation of a "cloudburst" was not advanced by any one who studied the phenomena at first hand, so far as the writer knows.

This paper has evoked much discussion, both favorable and unfavorable. The writer wishes to acknowledge a debt to all who discussed it, whether in agreement or in objection. The discussion has furnished important data, and thrown much light on various matters connected with these floods.

There is no controversy as to the occurrence of large cross sections or high velocities. Those familiar with the ground, who discussed the paper, found similar facts. There is definite disagreement as to causes, however, and as to runoff conditions during the flood. Objections to the explanation given by the writer have practically all revolved around the proposition that rate of runoff never exceeds rate of rainfall, except in certain well known and simply explained cases, such as the failure of dams.

The writer has no doubt that all of the factors enumerated in the objections were present during the flood, to some extent, or at some time. It is not believed, however, that any of them, or all of them combined, were sufficient to account for the destruction wrought by the flood, or for the other facts disclosed.

The discussion has revealed three basically differing explanations to account for the observed phenomena:

(1) That the large cross sections resulted from a retardation of the stream flow as it passed down the channels, and its consequent accumulation, this retardation being caused by a mass of water and debris, which acted somewhat as a moving barrier, traveling downstream at a slower rate than the velocity of the water behind it, and gradually impounding such water;

(2) That these flows were slow-moving bodies of water, very heavily loaded with debris and advancing somewhat as a mud flow, and that the rate of flow

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[53] Cons. Engr., Los Angeles, Calif.

of water in the streams was never greater than would be accounted for by the rate of rainfall; and

(3) That the cross-sectional areas of channels were temporarily greatly reduced by accumulations of debris. Over a part, or all, of the widths of the debris, streams of water flowed at rates of discharge that could be explained at all times by the rates of rainfall on the drainage area. These streams might attain high surface velocity. Subsequent cutting removed most of the temporary channel loads. As a result of these conditions, cross sections measured subsequent to the flood did not correctly represent the situation that existed during the flood.

The first explanation is that adopted by the writer. Waves were constantly augmented by high-velocity surface water from upstream which advanced to the wave front and then largely dissipated its energy in turbulent vertical action upon the stream beds.

Although the data do not permit an estimate of the instantaneous flow, perhaps a rough approximation to the lower limit of flows directly behind wave fronts is feasible. This involves the surface velocities as computed from splashes.

In Hall-Beckley Canyon, surface velocity has been computed as not less than 25 ft per sec near the left bank of the stream. If clear water, only, had flowed in this canyon, the properties of the stream bed would require a hydraulic radius of 2.93 ft in order for the surface water to reach this velocity. At this point in the canyon, a section of 265 sq ft is reached before the hydraulic radius becomes 2.93 ft. For such a cross section the flow of clear water needed to sustain this surface velocity is 5,300 cu ft per sec, equal to a runoff over the drainage area at the rate of 11 in. of depth per hour. When allowance is made for the heavy debris load, then the quantity of water flowing is increased by the quantity necessary to impart velocity to the debris.

The second explanation proceeds upon assumptions of average velocity and ratio of volume of water to debris which have been given variously by different writers. The following are about as low as were given by any one: Average velocity = 10 ft per sec, and ratio of water = one third of volume.

On this basis the rate of flow of water and debris above tops of check dams in Hall-Beckley Canyon becomes 6,800 cu ft per sec, and clear water is 2,266 cu ft per sec. This is at the rate of almost 5 in. in depth per hour over the drainage area.

The measurements at this point did not seem to be open to question. It does not appear to the writer to be possible to invoke the second explanation in Hall-Beckley Canyon, even when average rate of flow and percentage of clear water are used, which are lower than the most extreme values that any one has suggested. This same statement applies to Winery, Mullaly Fork, West Fork of Eagle, Shields, Dunsmuir, Cooks, and Blanchard canyons.

The third explanation calls for positive physical elevation of the stream beds from 6 to 20 ft throughout almost the entire length of the channels. These fills are stated to be formed during the earlier stages of the flood, when waters are rising, and to be composed of boulders, gravel, sand, and fines. The fills

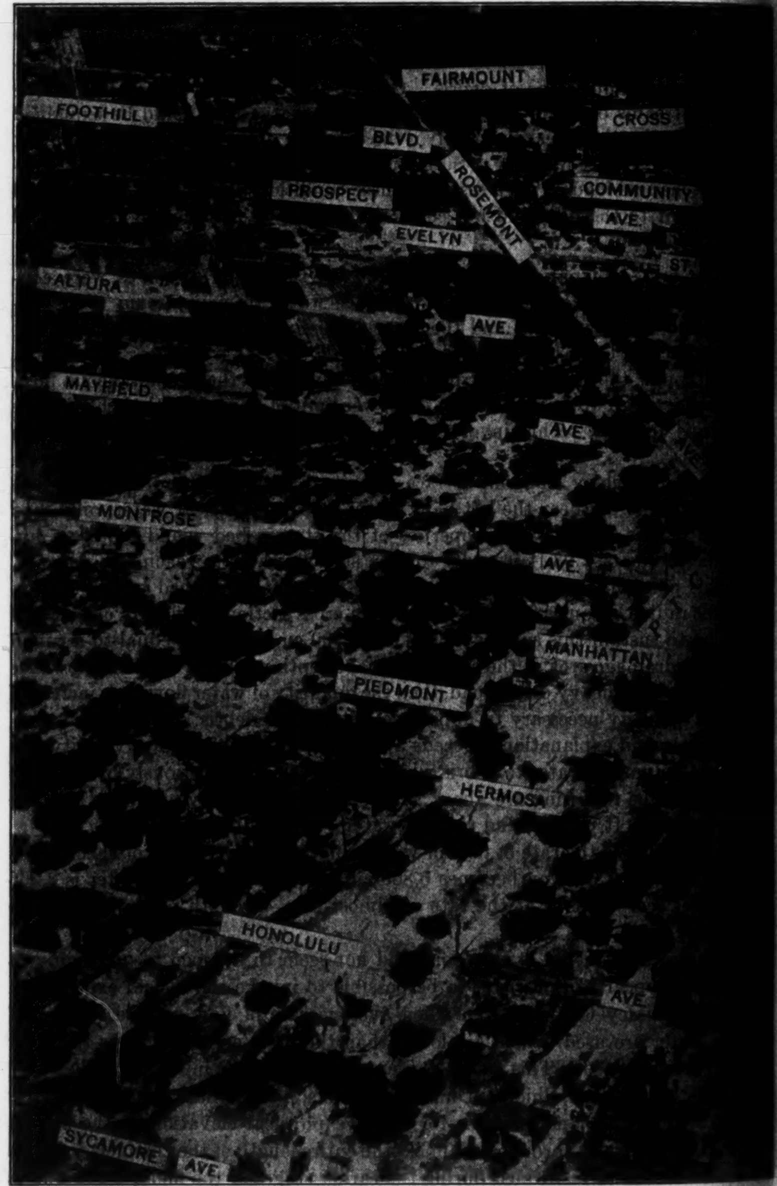
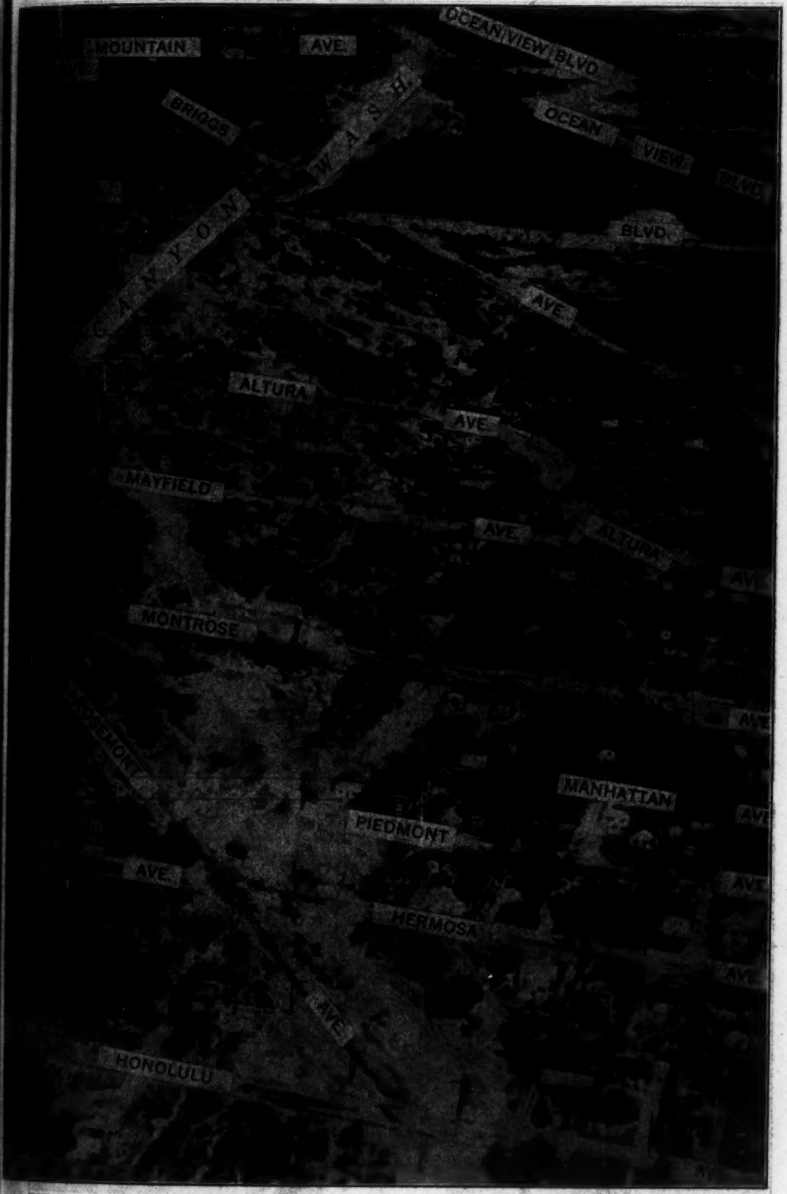

FAIRMOUNT
FOOTHILL
CROSS
BLVD.
ROSEMONT
PROSPECT
COMMUNITY
AVE.
EVELYN
ST.
ALTURA
AVE.
MAYFIELD
AVE.
MONTROSE
AVE.
MANHATTAN
PIEDMONT
HERMOSA
HONOLULU
AVE.
SYCAMORE
AVE.

Fig. 22.—Path of Destruction and


MOUNTAIN
AVE.
OCEAN VIEW BLVD.
BRIGGS
WASH.
OCEAN
VIEW
BLVD.
CANYON
BLVD.
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ALTURA
AVE.
MAYFIELD
AVE.
ALTURA
AVE.
MONTROSE
ROSEMONT
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HERMOSA
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HONOLULU
AVE.

Overflow from Pickens Canyon

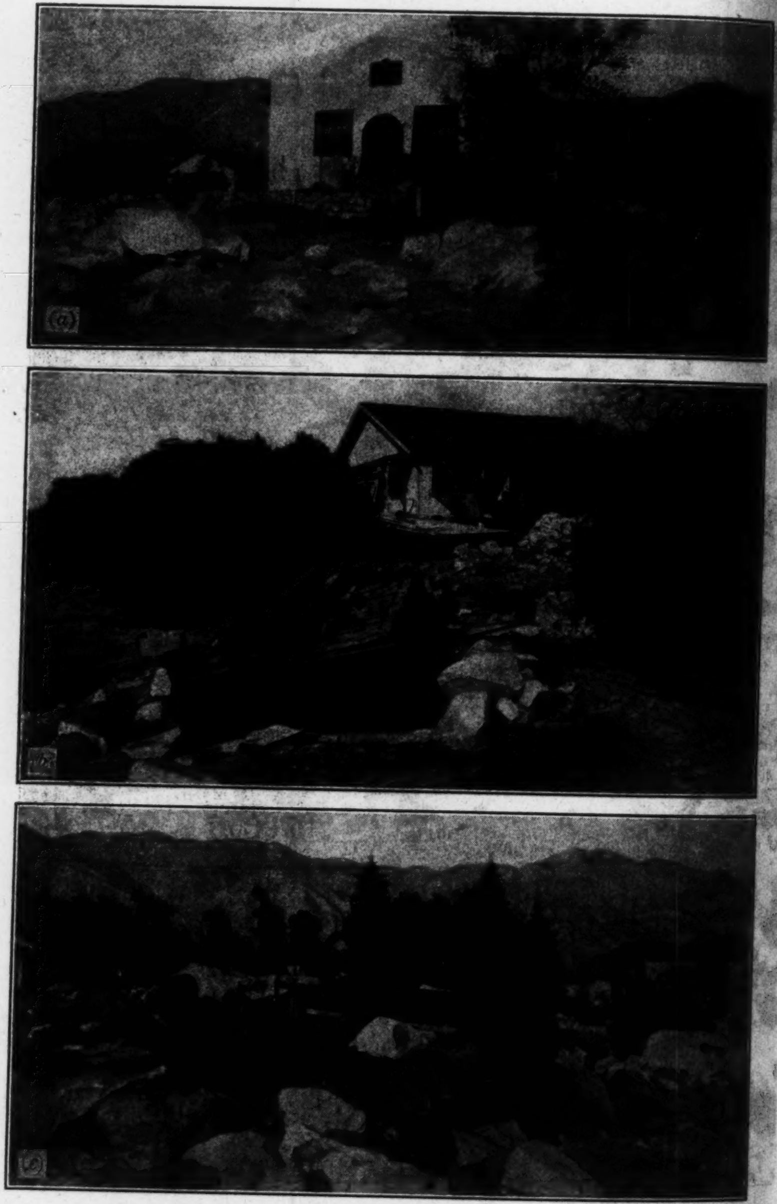

FIG. 23.—BOULDERS AND DESTRUCTION LEFT BY OVERFLOW OF PICKENS CHANNEL

are scoured out at a later stage of the same flood, leaving, however, numerous evidences of their former presence. The writer saw no places or photographs to indicate that this had occurred except in a few fills in protected locations within the channel. These were never more than 3 ft deep, and were not typical of usual conditions. Fig. 15 was from a photograph taken after an ensuing small flood in October, 1934,[54] almost a year later. It does not relate to the type of action described, but to one almost diametrically opposite. This fill was composed of almost pure sand, about 5 ft deep. It was made at a late stage of the January flood, when velocities were so low that the flow carried only sand, and deposited a large part of that. Afterward it was scoured out by the rising water from a subsequent storm. The same thing occurred at that time in many places in the lower channels. It can be duplicated whenever a small flood follows a major one. The early stages of a large flood scour out all evidence of these deposits of loose sand.

On the inhabited slopes below the mountains many of the difficulties of the second and third explanations are common to both. Fig. 22 shows the cone below Pickens Canyon and the widespread destruction from this flood. The catchment area is 1.81 sq miles. Any satisfactory explanation must account for conditions here as well as in the canyons. Fig. 23 shows boulders thrown outside of Pickens channel downstream from the mountains, in the flood of January 1, 1934. Figs. 23(*a*) and 23(*b*) are views on the east side and Fig. 23(*c*) is a view on the west side. The boulder in the foreground of Fig. 23(*a*) weighed about 13 tons. If either the second or the third explanation is correct, such boulders were deposited, together with gravel, sand, and fines, by comparatively small streams of water. Later in the course of the flood, underloaded streams of water, also comparatively small, are supposed to have overflowed from the channel and washed all of the gravel, sand, and fines from each of these areas.

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[54] "Flood in La Cañada Valley, California, January 1, 1934," by Harold C. Troxell and John Q. Peterson, U. S. Geological Survey *Water Supply Paper No. 796–C*, 1937, Plate 24–A.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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Paper No. 2104

BRIDGE AND TUNNEL APPROACHES

By John F. Curtin,[1] Jun. Am. Soc. C. E.

With Discussion by Messrs. Dean G. Edwards, John W. Beretta, George Hartley, George H. Herrold, Park H. Martin, Charles M. Noble, and John F. Curtin.

Synopsis

Between 25% and 50% of the total cost of vehicular bridges and tunnels is expended on their approaches; and yet relatively little of the diligent research and analysis which accompanies this expenditure has been made available to the engineering profession. In this paper an attempt is made to review the more significant elements of bridge and tunnel approaches and to make suggestions for their design. The scope of inquiry comprehends three types of approaches: Direct extension, reservoir, and tapered plaza with feeder connections. These are reviewed summarily, with a brief comparison of their relative merits and disadvantages.

The development of approach connections is outlined from the requirements of traffic and the adequacy of existing routes. In addition, the requirements for design speed, sight distance, curvature, and other alinement features of approaches are reviewed. The design of bridge and tunnel plazas is considered, based upon their functions of toll collection and lane convergence, and a discussion of decentralized plazas, reservoir space, layout of plazas, and portal transitions is included.

In this paper particular attention is given to the toll structure because of the increasing use of the toll type of river crossing and the more stringent requirements which are made of it. The kinetics of vehicles is the paramount consideration throughout, for it is believed that successful operation is dependent entirely upon smooth, uninterrupted traffic flow.

Definitions

Bridge (and tunnel) approaches have been defined by the late Wilbur J. Watson, M. Am. Soc. C. E., as follows:[2]

Note.—Published in November, 1939, *Proceedings*.

[1] Highway Engr., Economist, Am. Petroleum Inst., New York, N. Y.

[2] "George Washington Bridge: Approaches and Highway Connections," by J. C. Evans, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 97 (1933), p. 436.

> "The approaches to a bridge comprise the traffic arteries leading to the ends of the bridge proper, and such adjustments of alignments and grades of said arteries in the immediate vicinity of such ends as is necessary to afford the maximum convenience of access, and render available to the public the entire capacity of the bridge proper."

A considerable variety of approaches has been developed to secure the interchange of traffic which this definition implies, but, for review purposes, they may be classified into three broad groups as follows: Direct street or highway extension, reservoir, and tapered plaza with feeder connections. These types are not sharply defined, the difference being mainly in principle of operation. The approaches to many existing river crossings embody the characteristics of more than one group and some fall beyond these classifications entirely, but for the most part, the methods of operation outlined herein are applicable to most terminal facilities.

Direct Street or Highway Extension.—Because of its simplicity, the direct street or highway extension is still the most commonly constructed type of bridge or tunnel approach. Thousands of bridges are in use throughout the United States on which the connecting roads lead directly to the structure proper. In most cases, the river crossing is regarded as an integral part of a particular traffic route, rather than one which is to link several routes at each end, and for this reason the design requirements on its approaches are the same as those on the route which it connects. Little planning beyond a consideration of the maximum gradients in bringing the bridge or tunnel roadway to grade is required. Whether tolls are collected or not makes little difference, for the volume of traffic seldom requires more than a slight widening of the roadway through the toll collection area in order to provide space for the booths.

This type of approach has been well applied in rural areas, even to the ends of large structures, one example being the Marin County approach to the Golden Gate Bridge in California. Where two or more highways converge toward a river crossing, the direct extension has been modified in some cases with a traffic circle, thereby eliminating the bothersome left turn and enabling the vehicles to move steadily to and from the structure.

In cities, however, the direct street-extension approach has definite shortcomings. A bridge or tunnel is designed to handle continuously moving traffic and can accommodate approximately 1,500 vehicles per hr on each lane, whereas a signal-controlled street has intermittent flow with a lane capacity of from 500 to 800 vehicles per hr. Consequently, an approach that connects directly to a signal-controlled street materially reduces the capacity of the structure. Even with two or three streets at the end of an approach, the cross movements and turning movements of bridge or tunnel traffic, together with local traffic, hinder the access to the structure and limit its lane capacity to from 750 to 1,000 vehicles per hr.

Traffic circles have been utilized to overcome this disadvantage in undeveloped sections of cities, for they enable vehicles to approach the structure from two or more arteries continuously. The Goethals Bridge approach in Staten Island, New York, and the Rockaway Beach approach to the Marine

Parkway Bridge on Long Island, New York, have rotary connections between the approach proper and the connecting roads. This application of the traffic circle is limited to outlying districts, however, because the large part of the unused area in the central island makes rotaries uneconomical in the highly developed part of a city.

The direct street-extension approach develops additional complications on a toll structure, if the collection booths are not placed a sufficient distance beyond the intersection of the approach and the signal-controlled street. The vehicles enter the approach in surges, according to the traffic-light cycle, and the collectors are not able to pass all the cars that move toward the booths on each interval. Consequently, traffic accumulates back across the intersections and blocks the flow of vehicles on the intersecting street.

On the approach itself, off-bound vehicles are required to wait in long queues which sometimes extend back on the structure proper. This condition in undesirable on a bridge, and dangerous in a tunnel. The hazard which may result is illustrated by an experience in the Liberty Tunnel in Pittsburgh, Pa. One evening a tie-up beyond the approach caused a line of stalled vehicles to form back through one of the tubes. Before the policemen could order all the motors to be turned off, the tunnel was filled with carbon monoxide and, although no fatalities resulted, several people were overcome by the gas.

Reservoir Approaches.—To obviate the disadvantages of the street-extension approach, the engineers of the Holland Tunnel in New York, N. Y., developed a reservoir for tunnel traffic (Fig. 1) by clearing the block in front of the portal. Instead of forcing traffic to "worm" its way through a street that crosses the tunnel roadway at grade, they paved the entire block, thus enabling the vehicles to enter the tunnel from four street intersections. By the use of painted lines and channelizing islands they established the bounds of the plaza and directed the movements of the vehicles entering it.

This type of plaza provides a storage space or reservoir wherein 150 to 200 vehicles from the bordering streets can collect without obstructing local traffic. The capacity of the facility is not limited by the traffic-light cycle on any one of these streets, because the alternating green signal permits north and south traffic to enter while those from the east and west wait, and vice versa. On a reservoir plaza, the toll booths are arranged in a circular arc to accommodate the vehicles which enter the plaza at all angles. Consequently, the bridge or tunnel is not wholly subjected to the intermittence of local traffic movement and it is able to accommodate more than 1,200 vehicles per lane in peak hours.

The reservoir approach has been principally applied to tunnels because it permits cars to clear through the tubes and wait on the plazas in the event of an emergency, such as a traffic tie-up. This type of approach is used at the Mersey Tunnel in England, both at the Kingsway entrance in Liverpool and the King's Square entrance in Birkenhead. The over-all shape of these plazas is a sector, with the portal at the apex and the reservoir at the widest part. The entrance and exit facilities are combined on each plaza, and the toll booths are arranged on one quadrant of the semicircular opening. The Antwerp Tunnel under the Schilde River also has a reservoir plaza on one side.

The primary disadvantage of the reservoir approach is that at peak periods it causes all vehicles to stop and wait, or to move slowly and cautiously, before entering the tunnel. Practically all of these approaches are of such shape as to preclude the continuous movement of vehicles into the tunnel after they leave the toll booths. The lanes leading to the portal all converge on one junction together and the angles between these lanes are too great to permit smooth convergence. On the New York entrance plaza of the Holland Tunnel, the twelve lanes are distributed through an angle of 180°. Traffic on the outside lanes makes a right-angle turn to enter the tunnel, a movement which slows down its convergence with the other vehicles. In addition, the twelve radial lines of cars must narrow into two lanes in a relatively short space. During periods of heavy traffic flow, an officer is needed on this type of plaza to direct the entrance movements.

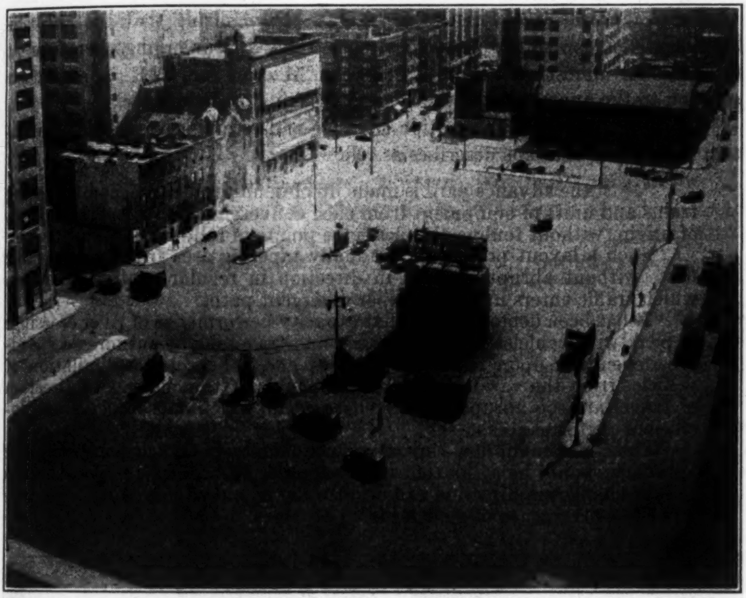

FIG. 1.—THE RESERVOIR PLAZA AT THE NEW YORK CITY ENTRANCE OF THE HOLLAND TUNNEL, SHOWING THE SEMICIRCULAR ARRANGEMENT OF TOLL BOOTHS

In peak hours, this unwieldy maneuvering often causes traffic to overflow the plazas and collect on the bordering streets. As many as 450 vehicles have been observed waiting to enter the Holland Tunnel, 250 of them off the plaza and obstructing local traffic. Although this condition is likely to occur on any facility in which the volume exceeds the capacity, in this instance it is the inertia of the convergence movements which impedes the flow of traffic and limits the capacity of the tunnel.

Another disadvantage of the reservoir plaza is its ill-adapted use of storage space, particularly on the square or rectangular plazas. Even when crowded, only about 60% of the reservoir area is usable for storing cars. In the business or industrial section of a city, it is uneconomical to waste such a large percentage of available area.

Tapered Plazas with Feeder Connections.—Recognizing that the railroads had been coping with the problem of converging and diverging operations at terminals for almost a century, J. C. Evans, M. Am. Soc. C. E., applied their solution to the approaches of the George Washington Bridge.

The plazas became combination advance and receiving yards, the adaptation of which Mr. Evans describes as follows:[3]

> "* * * the advance yard is made up of definite tracks, accommodating trains and units of equipment from each converging line or common point of origin, without fouling the clearance points of following traffic.
>
> "Such a layout provides a gradually tapering area from which traffic flows without abrupt changes in direction in regular routine, and into which traffic enters upon its proper divergent path.
>
> "It has been demonstrated that capacity performance of river crossings in peak with vehicles properly spaced and travelling at authorized speeds are dependent upon entrance facilities and the proper assimilation of vehicles from the various lanes of approach. If the changes in direction of travel are too abrupt, interferences occur which cause delays and inefficient performances.
>
> "Since maximum flow-capacity depends upon entrance conditions, the approach should be so located with reference to various accesses that no abrupt changes in direction can occur upon it, but should permit the progressive convergence of vehicles from all lanes at acute angles."

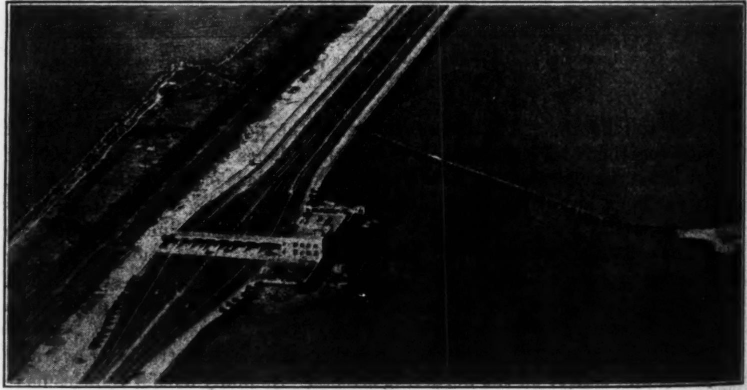

FIG. 2.—THE DIAMOND-SHAPED TOLL PLAZA OF THE SAN FRANCISCO-OAKLAND BAY BRIDGE

The efficacy of this analysis led to an almost universal adoption of the tapered plaza. Following its institution on the George Washington Bridge, it was constructed on the Golden Gate Bridge and the San Francisco-Oakland Bay Bridge (Fig. 2) in California, and in the Lincoln Tunnel in New York City.

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[3] "Approaches to River Crossings," by J. C. Evans, an address at Harvard University, May 13, 1938 (not published).

To augment the utility of the tapered plaza, Mr. Evans extended a series of direct feeder connections to the main traffic arteries whereby the bridge traffic was collected and diffused in its line of travel without interefering with local traffic. In cities this method of infiltration to the street system is particularly advantageous because it distributes the volume of traffic over a wide area and accommodates the through movements in the various directions of travel. On the New Jersey approach of the Lincoln Tunnel (Fig. 3) an express highway has been extended westward for 1.5 miles in New Jersey through Weehawken, Union City, and North Bergen with separate ramps for inbound and outbound traffic in each of these communities. Similarly, the San Francisco-Oakland Bay Bridge accommodates its east bay traffic by extending a direct connection north for two miles and another east for one mile from the bridge. These connections not only facilitate traffic in these directions, but liberate the streets in the vicinity of the approach from undue congestion.

This type of approach, the tapered plaza with feeder connections, satisfies the previously cited definition by Mr. Watson[2] most completely because it affords the maximum convenience of access and makes the entire capacity of the structure available to traffic. It is also the most expensive type, since it involves a far greater amount of construction and land acquisition than is required for either a direct extension or reservoir approach. The expenditure warranted to obtain maximum convenience and availability of capacity on the approaches of a bridge or tunnel depends upon the volume of traffic that is anticipated to use the facility during its service life. In this paper, however, the scope of the subject and the limitations of space preclude any detailed discussion of the economics of approach planning, except as it is linked with the discussion of approach connections and plazas which follows.

APPROACH CONNECTIONS

All the major streets and highways in the vicinity of a proposed river crossing are potential feeders of traffic to it. One of the primary aims in locating the approaches of a structure is to secure the position that is best suited to the collection and diffusion of vehicles by way of these arteries; and this includes not only existing street or highway systems but any proposed improvements, since the latter may become the heaviest feeders of the structure.

In developing an approach, the objective is to provide a system of well-selected, short trunk connections which will tie these feeder streets and highways to the bridge or tunnel. Each connection should have a complement of branching laterals, placed strategically to the zones of origin-destination. Two criteria determine the development of a system of approved connections: (*a*) The adequacy of the street or highway system; and (*b*) the expenditure warranted in providing time and distance savings to the motorist.

The first criterion indicates the capability of the existing system to feed and diffuse the traffic of the river crossing, whereas the second determines the degree to which improvement of this system is justified. If a toll bridge or

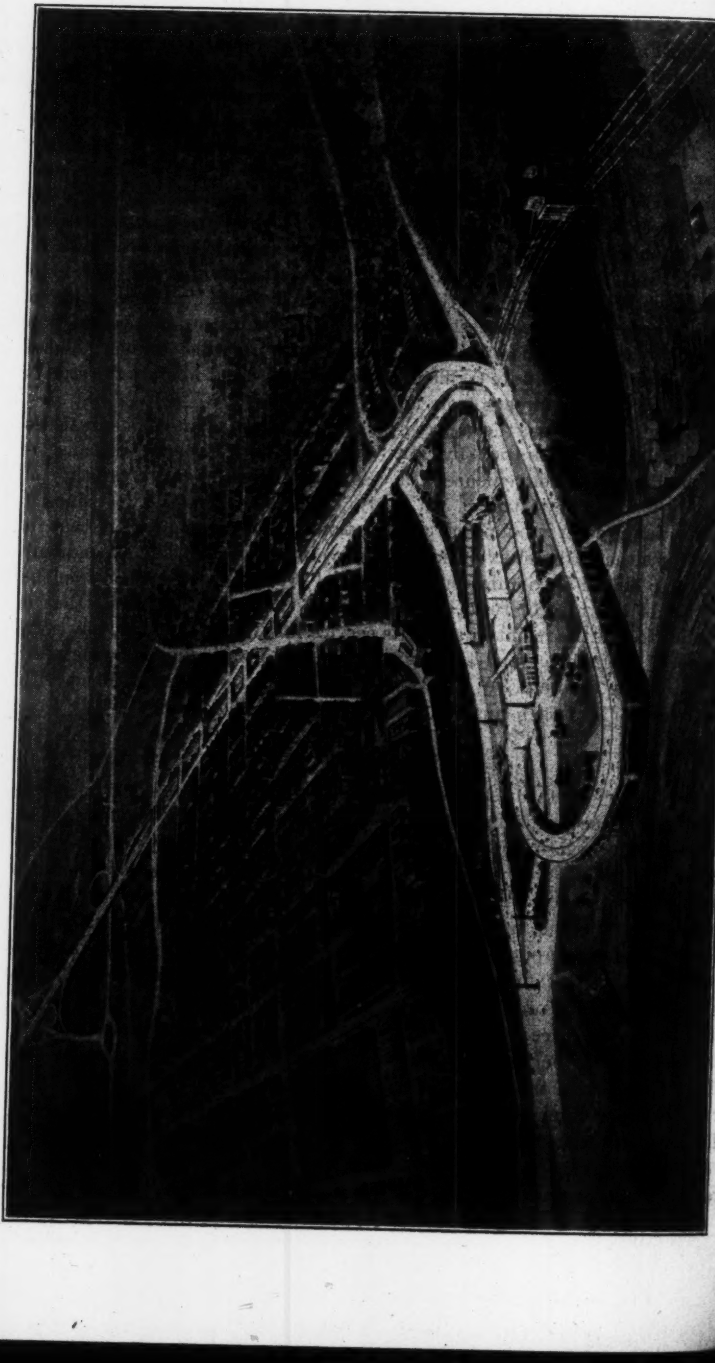

Fig. 3.—The New Jersey Approach of the Lincoln Tunnel Showing the Express Highway Connection Which Passes Over and Under the Local Streets but Connects with All the Main Routes

tunnel is to be utilized fully, its connections should be able to accommodate all of its traffic adequately and, in addition, provide benefits to the motoring public commensurate with the toll.

In developing the fantail of connecting links between the structure and the zones of origin-destination it is necessary, therefore, to:

(1) Determine the geographical distribution of the traffic over the existing streets and highways contributing to the proposed crossing, and the adequacy of these routes to accommodate this additional traffic;

(2) Develop the connections that are necessary for the proper dispatching of the traffic at the crossing at the time it is opened to service; and

(3) Make provisions for the expansion of these approach connections to handle a future increase in traffic and to embrace any proposed highway developments in the vicinity of the structure.

Geographical Distribution of Traffic.—From the origin-destination studies of the traffic on existing river crossings and the determination of the volume that can be diverted to the proposed crossing, a flow diagram can be prepared which will indicate the percentage of traffic terminating in each zone. With this diagram as a basis, and assuming that traffic follows the most direct routes between the structure and the zones, it is possible to select the existing streets and highways over which the various portions of bridge or tunnel traffic will be diffused. This is essentially the same procedure that is used in estimating the time and distance savings in the determination of diverted traffic. If two or more routes are available in any line of travel, a distribution is made among them in anticipation of the motorist's choice. This distribution pattern suggests the routes that are to become the main arteries of traffic to and from the proposed crossing and focuses attention on the connections that should be developed or improved.

A survey is then made on these feeder routes to determine the gross hourly capacity of each and the volume of traffic that uses it normally. In planning the connections of the Lincoln and Queens Midtown tunnels in New York, estimates[4] were made of the lane capacities of all Manhattan streets in the vicinity of each tunnel plaza, appropriate reductions being made for the 90-sec traffic-light cycle, parking at curb lanes, and elevated transit line columns. The one-way crosstown streets were found to have a working capacity of 550 vehicles per hr, whereas the capacity of the avenues ranged between 950 and 1,530 vehicles per hr in each direction. In addition, a volume count was made on each of these streets for the hour of the average week-day during which the tunnel traffic was predicted to be heaviest. The difference between the working capacity and the volume during that hour represented the remaining capacity available to handle tunnel traffic.

The remaining capacity of each of the approach streets and highways is a primary factor in the planning of connections because it is a measure of their capability to absorb the traffic over the crossing. By combining these data with the proportional share of bridge or tunnel traffic that is estimated to use each route, the points of high concentration and possible congestion may be determined. These locations can then be improved, or connections to alternate routes developed, so that the accessibility of the crossing will be maintained at all times. In numerous cases, the arteries contiguous to the plaza have been given too little attention, with the result that traffic has backed on to the plaza during peak volume periods. For this reason, it is necessary to consider every point of high traffic concentration in the existing system which is to be used for approach and to make the proper deductions for every obstruction to free flow on these feeders. The potential traffic of a vehicular crossing can only be developed when there is sufficient capacity on its approach roads to distribute it adequately.

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[4] "Report of Probable Traffic Requirements of Queens Midtown Tunnel and Its Arterial and Approach Street and Highway Requirements," by Nathan Cherniack, Assoc. M. Am. Soc. C. E., September 30, 1937.

Development of Connections.—Knowing the peak-hour volume in each general direction from the flow diagram and the capacity of arteries available to this traffic, the next step is to develop a system of laterals branching from the plaza which will facilitate the movement of this traffic. Essentially, it is an hydraulics problem of fitting wye, tee, and elbow connections between a high-pressure main and a system of service lines.

In rural or suburban areas, the consideration of sufficient available capacity of existing routes to absorb the traffic from the crossing is not so much of a problem as that of direct, expeditious connection. Greater benefits to the potential bridge or tunnel patronage in time savings, distance savings, and convenience, can be afforded here and it is desirable to focus attention on this phase of the approach development. A larger and more extensive network of connections can be provided which will draw traffic from all important arterial routes. The New Jersey approach of the Lincoln Tunnel (Fig. 3) is an example of a fast, convenient system of connections which converges all the nearby roads to the tunnel.

In large cities, however, the cost of real estate and the interference with other structures generally prohibits the provision of any but the minimum number of separate connections required to "nose" each vehicle in its proper direction. The objective is to distribute the fixed crossing traffic on the streets in the vicinity of the plazas in accordance with the geographical distribution as previously determined and with due regard to the available margins of street capacity in each direction.

In the studies of the Manhattan approach of the Queens Midtown Tunnel, the peak-hour volume was apportioned to four quadrants—northwest, northeast, etc.—from the flow diagram and then distributed among the streets intersecting the plaza avenues. The surplus margin of street capacity was determined for each quadrant, and although the minimum requirements were not definitely established, the designers were afforded a measure of the adequacy of these connections to serve the tunnel traffic.

The Manhattan approaches of the Lincoln and Queens Midtown tunnels are particularly well adapted to the gridiron street system, because they both provide approach avenues that intersect the crosstown streets (Fig. 4). In developing these approaches, the designers were limited by high property costs to a distributer avenue, from each plaza, of sufficient length to accommodate the collection and diffusion of vehicles without congestion in the area contiguous to the plazas. Each avenue is precisely long enough to intersect two or three cross streets in each direction and thereby to collect and diffuse the tunnel traffic without exceeding the margin of capacity on any of them. This system is flexible because the motorist has a choice of two or three streets in each direction. If at any time their total margin of capacity should become insufficient to handle the tunnel traffic, the distribution avenues can be extended to intersect additional cross streets.

On the Oakland approach of the San Francisco-Oakland Bay Bridge, the spread of traffic into its lines of travel is accomplished by a branching pattern of short limited-way structures which convey each part of traffic directly to an arterial route (Fig. 5). This differs from the diffusion system of the Lincoln and Queens Midtown tunnels in that it does not distribute each line of travel into the street system multifariously, but conducts it to a main artery in its direction. When it can be afforded, this system of separate connections for each line of travel is more advantageous because it eliminates any possibility of congestion due to cross movements and overlapping of traffic near the plaza. In addition, it offers the advantage to motorists of greater saving in time due to its freedom from the street-signal system.

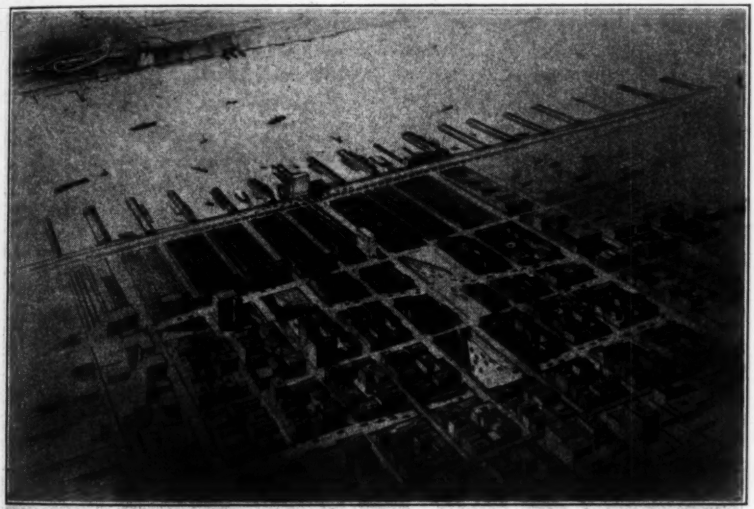

FIG. 4.—THE NEW YORK CITY APPROACH OF THE LINCOLN TUNNEL, SHOWING THE TAPERED PLAZAS AND THE TWO APPROACH AVENUES WHICH COLLECT AND DIFFUSE TRAFFIC TO THE CITY STREETS

Numerous combinations of existing street and express connections are possible, the development of each being a matter of economic study. If the volume in any single direction warrants it, a direct freeway connection to the main route in that direction should be constructed. Wherever existing streets are to be used for connections, however, an adequate margin of capacity must be made available to bridge or tunnel traffic either by widening these streets or by embracing more of them into the approach system.

Provisions for Future Connections.—The limitations in traffic forecasting make it impossible to plan the approach connections beyond the first few years of operation. Nor is it possible to predict which of the extant proposals for street or highway traffic improvement in the vicinity of the structure will materialize. These uncertainties make it necessary to provide latitude for future expansion of the approach connections.

Although the proposed routes of a street or highway development program are important considerations in locating an approach, they become paramount factors in the planning of its connections. The location of a bridge or tunnel will often influence the conduct of a local development program. Therefore, some provisions must be planned to incorporate these arteries into the approach system if its utility as a transportation facility is to be maintained. In all of the planning studies for the New York approach of the Lincoln Tunnel, provisions for direct connections to the proposed 38th Street Crosstown Tunnel were included, because traffic studies indicated that 25% of its patronage would use the latter facility if it were constructed.

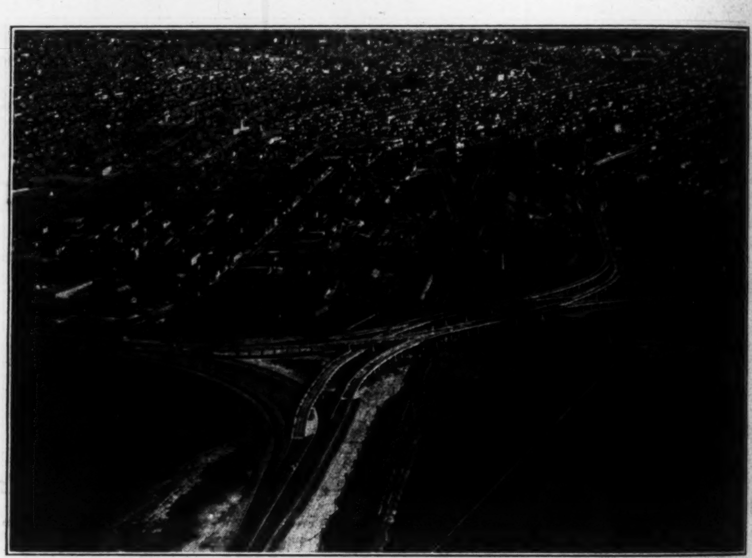

FIG. 5.—THE DISTRIBUTION STRUCTURE AT THE EAST BAY END OF THE SAN FRANCISCO-OAKLAND BAY BRIDGE WHICH COLLECTS AND DIFFUSES TRAFFIC TO THE CITIES OF BERKELEY, ALAMEDA, AND OAKLAND, CALIF.

The increase in traffic on both the river crossing and local streets is another factor which cannot be predicted, but must be anticipated. In determining the adequacy of local streets to handle bridge or tunnel traffic only the present margins of capacity are calculated. It is essential, therefore, that some real means of relief be planned in the event that these streets become congested by bridge or tunnel traffic in the future. Recognition of the possibility of such a condition led to the inclusion of a provision in the agreement between the City of New York and The Port of New York Authority for the construction of an approach tunnel under 178th Street when the annual volume of George Washington Bridge traffic reached 10,000,000 vehicles. It was estimated that the local streets between the bridge plaza and the eastern side of Manhattan would not accommodate the proportional share of traffic in that direction in excess of that amount. By deferring the opening of this connection until it is needed, considerable savings are made and the surplus revenues from operation of the bridge are available to finance the improvement.

Types of Connections.—In this paper it is not possible to discuss all the possible street and highway improvements, relocations, and new developments which may be utilized in a system of approach connections. For rural areas, the standards of the American Association of State Highway Officials indicate modern practice and they are the best guide. No similar standards are available for urban connections but there are numerous possibilities, ranging from the widening of local streets to the construction of express highways.

Two general methods of handling traffic in cities have proved sufficiently effective to warrant consideration of them for connections to bridges and tunnels. These are the steady flow system, developed by Fritz Malcher, and the freeway as expounded by Miller McClintock.

The steady flow system[5] consists primarily in the elimination of signal control, left turns, and right-angle intersections by an application of the rotary principle. A series of elongated islands, 10 to 40 ft in width and 200 to 500 ft in length, form a medial strip along the center of the roadway and separate the vehicles moving in opposite directions. The islands extend entirely across each intersection, thereby preventing any vehicles from crossing the roadway at right angles, or from making a left turn. A motorist who wishes to cross the road, or to go left on it, must turn right into the main traffic stream and weave across it diagonally until he reaches the turning roadway between the islands. After passing through the medial strip, he enters the stream in the opposite direction by a left turn and can then continue in this direction or turn off on the intersecting street. All vehicles crossing a steady flow connection must make this U-turn on it for half a block or more.

The steady flow system is applicable to streets that are being widened or improved to accommodate a large volume of bridge or tunnel traffic, in addition to local traffic, and it is particularly useful when there is not much cross traffic. Its exponents claim that it increases the capacity of each lane on the roadway 132% over that under the signal-control system; and there is little doubt that traffic can move faster when it is freed from the delays caused by stop-and-go signals. This system of traffic control is used effectively in connecting the Marine Parkway Bridge to Beach Channel Drive in Rockaway Beach, Queens,

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[5] "The Steady Flow System," by Fritz Malcher, Harvard Univ. Press, 1935.

N. Y. The East River Drive leading to the Triborough Bridge in New York City is a modification of the steady flow system, although it is controlled by traffic signals, and left turns are permitted at a few places.

The second type of approach connection, the freeway or express highway, implies new constructions, for it is generally depressed below or elevated above the existing street system. As described by McClintock,[6] it embraces the following physical elements:

1. "Complete and continuous physical separation of opposed streams of traffic by a medial strip.
2. "Elimination of direct access from abutting property to the roadway and restriction of entries and exits to a limited number of locations.
3. "Complete separation of grade at all intersections with no cross movements across the operating lanes of the freeway.
4. "A cross section design permitting adequate segregation of fast and slow vehicles, with accelerating lanes at the entries and decelerating lanes at the exits."

This type of approach connection is the ultimate in safe and expeditious travel because it delivers the traffic in a closed conduit past all conflicts and delays. In this respect, it is similar to the bridge or tunnel itself. Two outstanding examples of this type of approach connection are the extension of Grand Central Parkway to the Triborough Bridge (Fig. 6) and the express highway connecting U. S. Route 1 and Route 3 to the Lincoln Tunnel (Fig. 3). Both of these are flanked by streets on each side to accommodate local traffic and give access to abutting property.

Design of Approach Connections.—The modern car has been developed so completely that it is essential to provide for the utilization of its speed potentialities in the design of a new facility. In many cases, the justification of a new bridge or tunnel lies solely in its ability to offer a saving in time to motorists now using a ferry or congested fixed crossing. Whether this economic advantage is necessary or not, the pressure of modern traffic makes it necessary to consider the concept of time in the geometric development of the approaches to a facility.

Design Speed.—Because every feature of alinement is dependent upon it, design speed is the most significant factor in traffic design requirements. Design speed may be considered as that which will accommodate rapid, expeditious travel and provide an adequate factor of safety. It is the maximum uniform speed at which most drivers will operate on a highway. The Oregon State Highway Commission designs for what it terms a critical speed which is approximately 50% greater than safe operating speed; that is, the various details of alinement, curvature, superelevation, lane width, etc., of a 40-miles-per-hr highway are designed to provide safety for speeds up to 60 miles per hr. Whether this conception is practical has been questioned by several state highway officials. With the large mileage of straight alinement and the limited enforcement, highway speeds are usually determined by factors outside the province of road design. On bridges and tunnels, however, the mileage is

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[6] "San Francisco Citywide Traffic Survey," by Miller McClintock, 1937.

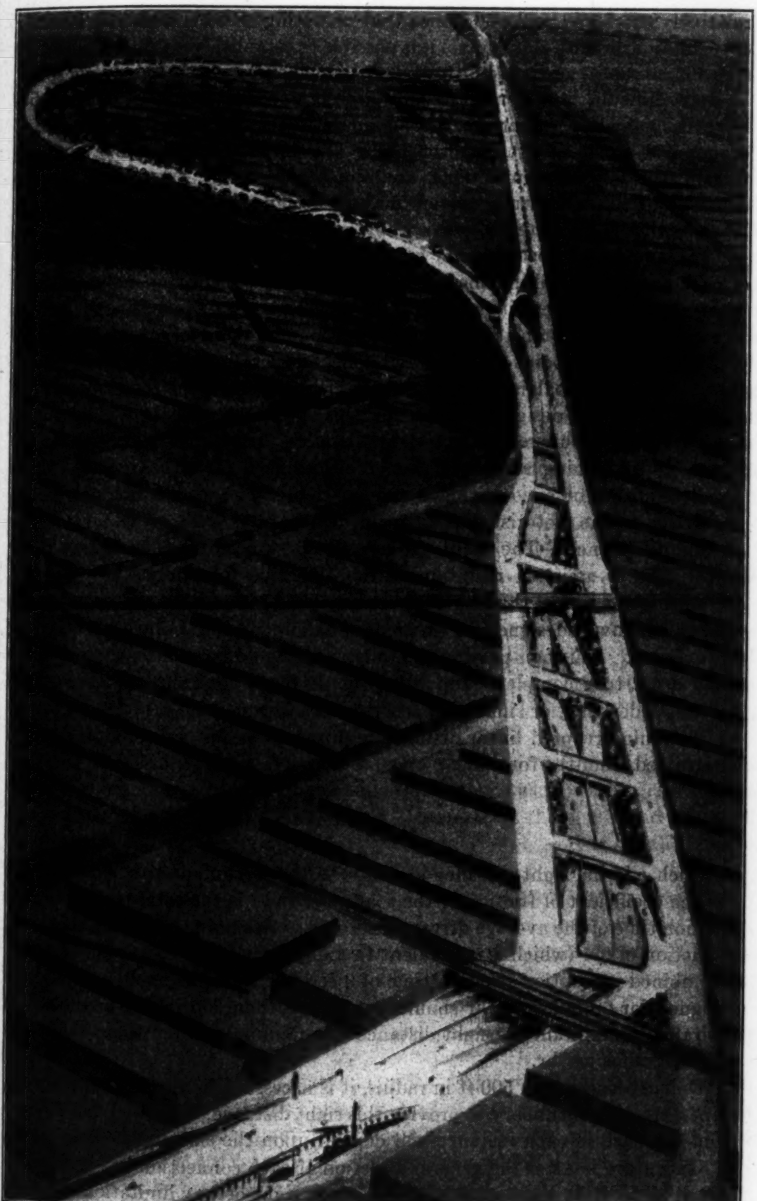

FIG. 6.—THE EXPRESS HIGHWAY CONNECTION FROM THE TRIBOROUGH BRIDGE TO THE GRAND CENTRAL PARKWAY IN QUEENS, NEW YORK

relatively short and well policed, so that operating speed is generally a matter of regulation. With this control it is possible to establish a maximum operating speed in advance, add a factor for safety, and thereby obtain the design speed.

It is difficult, however, to assume a design speed arbitrarily until studies have been made to ascertain the maximum operating speed for which it is economical to design the approaches. This will depend largely upon the terrain, the type of connections, and the nature of the surroundings. The connections to a city street system may be designed for only 30 miles per hr, whereas those of a state highway system are designed for 60 miles per hr. In general, the design speed should be great enough so that the operating speed of each connection will not be lower than that of the street or highway which it serves. Lower operating speed for the connection would cause it to function as a bottle neck, thereby producing congestion and delay.

From the standpoint of safety and efficient operation, it is more significant that the design be balanced so that every critical item of the roadway is calculated for the same design speed. Once this speed value has been determined, the curve radii, superelevation, sight distances, pavement widths, wall setbacks, and widening at curves should be designed for it. Any unexpected detail requiring or encouraging a sudden change of speed is not only a disconcertion to the motorist, but an accident hazard as well.

Sight Distance.—The ability to see ahead sufficiently is necessary if the motorist is to assist in maintaining the objectives of safety and efficiency. On a bridge or tunnel the length of unobstructed vision need not be as long as that on a highway designed for the same speed, due to the absence of overtaking and passing. This non-passing sight distance should be sufficient, however, to bring a car to a complete stop when the driver is confronted with a stalled vehicle, pedestrian, or similar obstruction in the same lane.

The uninterrupted linear distance required to permit safe stopping can be determined from the formula:

$$s = \frac{v}{30\,f} + 1.47\,t\,v \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which s is the sight distance in feet; v is the design speed in miles per hr; f is the coefficient of friction of the roadway; and t is the total perception and reaction time of the average driver, in seconds. Assuming a braking coefficient of friction of 0.4 (which is equivalent to a deceleration of 12.9 ft per sec^2) and a combined perception-reaction time of 3 sec, the sight distance on a roadway designed for 60 miles per hr should not be less than 565 ft. If possible, it is desirable to maintain the sight distance at a minimum of ten times the design speed in miles per hr.

On curves less than 500 ft in radius, it is necessary to recede any side walls, slopes, trees, or shrubbery to provide this sight distance. The setback distance from the curb in each instance will depend upon the sharpness of the curve.

Curvature.—After the general direction of each connection has been established, its location is determined chiefly from right-of-way limitations, controlling grades, and curvature. Knowing the design speed, the limiting curvature may be obtained from the formula:

$$r = \frac{0.067\, v^2}{E + f_l} \quad \text{(2)}$$

where r is the radius in feet; E is the rate of superelevation; and f_l is the coefficient of friction laterally.

Prevailing conditions in each instance will determine the applicable values of E and f_l. The superelevation can be limited to that which will counteract about three-fourths of the centrifugal force at the assumed design speed, and for practical reasons should not exceed 0.10 ft per ft. Tests to determine the friction that may be used with safety and without feeling side pitch outward have resulted in an acceptable friction factor of 0.15. These values substituted in Equation (2) result in a 430-ft minimum radius for a design speed of 40 miles per hr and a 965-ft minimum for a speed of 60 miles per hr.

On curves of 2,000 ft radius or less, transition spirals are necessary to soften the sudden application of centrifugal force on the vehicle. Otherwise the motorist will make the transition by cutting across the adjoining lane, thereby creating an accident hazard. The length of spiral long enough to ease the transition and gradually develop the superelevation may be computed from the formula:

$$L = \frac{3.16\, v^3}{C\, r} \quad \text{(3)}$$

in which L is the length of spiral in feet, and C is the rate of change of acceleration. Studies of riding comfort on street cars indicate that 2 is the most acceptable value of C. For a design speed of 40 miles per hr, the minimum length of spiral at the sharpest permissible curve is 235 ft; for 60 miles per hr, it is 350 ft.

Grades.—Grades are a controlling feature in the planning and design of a bridge or tunnel because they determine the distance which the structure must be extended inland to reach the surface. Likewise, on the plazas and approach connections, the grades determine the conformity of these parts of the facility to the topography and existing traffic arteries.

The prevailing low power ratio of trucks restrains smooth operation on steep grades, particularly when the facility is carrying a large volume of traffic. The following relationship has been developed by the Oregon State Highway Commission to express the operating speeds of heavy vehicles on ascending grades:

$$v_0 = 60 - 0.5\, W - 4.33\, G \quad \text{(4)}$$

in which v_0 is the operating speed in miles per hour, W is gross weight in thousands of pounds, and G is the percentage of grade. On the Pennsylvania Turnpike across the Allegheny Mountains, the maximum grade has been set at 3% in order that trucks may use this facility to its full capacity. Particular attention must be given to this factor, therefore, if the bridge or tunnel is expected to accommodate an appreciable volume of heavy vehicles.

With mixed traffic, it is desirable to limit the maximum upgrade on the plazas and main connections to 3.5%. If the plazas or connecting roads are separated for traffic in each direction, the downgrade maximum may be established at 4%. Depending upon the importance and length of other connections, these limits may be raised to 5% upgrade and 7% downgrade.

Lane Width.—Considerable study has been made on the width of highway lanes, and although no definite conclusions have been formulated, the indications point toward 10-ft lanes for light traffic roads and 11-ft or 12-ft lanes for heavy mixed traffic. These results are premised upon frequent passing of vehicles and speeds to more than 50 miles per hr, however—conditions which are not normally present on bridge or tunnel connections. Under the operating restrictions on these facilities, 10-ft lanes are normally adequate for safe clearance between all types of vehicles.

Due to the tendency of motorists to shy away from curbs, it is desirable to increase this width to 12 ft on the outside lanes of the connections when shoulders are not provided. The average edge distance, measured from wheel to curb, is 3.5 ft at 40 miles per hr, and it should be compensated in additional width. Similarly, on curves of less than 750-ft radius, the lanes should be widened to provide for the tracking of the vehicles.

Wye Junctions at Connections.—The critical points on every approach are the junctions between converging connections and between the connections and the plaza, for it is at these locations that fouling of converging or diverging streams of traffic is most likely to occur. Any point at which the roadways branch is a potential accident and congestion location, and it is worth while, therefore, to give it particular attention. Since the average motorist cannot be relied upon always to make the snap decision accurately, the choice between connections should be minimized at every junction and should afford sufficient time to the motorist to enable him to direct the vehicle smoothly.

For this reason, it is not desirable to have more than one roadway branching from another at any location. This should be especially guarded against at the plaza, where there is often apparent a false economy in branching off a group of connections together or at close spacing to each other. Adapting railroad location principles to the design of approaches, each connection should branch from the plaza singly and at sufficient distance from the others to be singularly distinguished by the motorist. Likewise, the network of connections should form a pattern of wyes, similar to the branches of a tree. The distance between junctions should be at least 300 ft (and preferably more) to enable the motorist to make the proper selection. This arrangement distributes the points of conflict and minimizes the hazard due to indecision or wrong decision at each junction.

The angle of convergence between roadways determines the smoothness with which the junction is made. It is desirable to reduce, as much as possible, the slowing down of traffic which results when streams of vehicles divide or join at the intersection of two roadways. Therefore, the connections should be designed so that the main part of the traffic follows a straight alinement while the minor part branches off at one small angle between it and the main roadway, or a right-angle turn, depending upon the design speed to be maintained and the prevailing physical conditions.

In so far as it is possible, a connection should give the motorist some indication of its ultimate direction before he steers into it. Frequently a roadway heads in one direction, and then turns through a 180° or 270° angle which is obscured at the outset. Where existing streets are used for approach connections, traffic is often required to make several turns, aggregating 360° or more, before finally heading in a particular direction. A certain amount of curvature is necessary to integrate the approach system properly; but it should be made visible or should be marked clearly so that the motorist has an indication of the direction in which each connection is heading. The distribution structure at the East Bay side of the San Francisco-Oakland Bay Bridge (Fig. 5), is an excellent example of a complex connection on which little confusion occurs. Despite its number of intertwined ramps, the motorist is always oriented, since each roadway in branching from a wye indicates its ultimate direction.

PLAZAS

A plaza is constructed at the ends of a river crossing for either of two purposes, namely, to collect or distribute traffic between the crossing proper and its approach connections, and to accommodate the collection of tolls. By recognizing the requirements of these two operations, the arrangement, dimensions, and other factors relating to the design of plazas may be determined. The extent of development in each application depends upon the volume and concentration of traffic that is expected to use the facility. On the Marin County approach of the Golden Gate Bridge, the plaza was omitted because all tolls are collected on the San Francisco side and there is only one major highway connected to the structure directly. In contradistinction to this, the

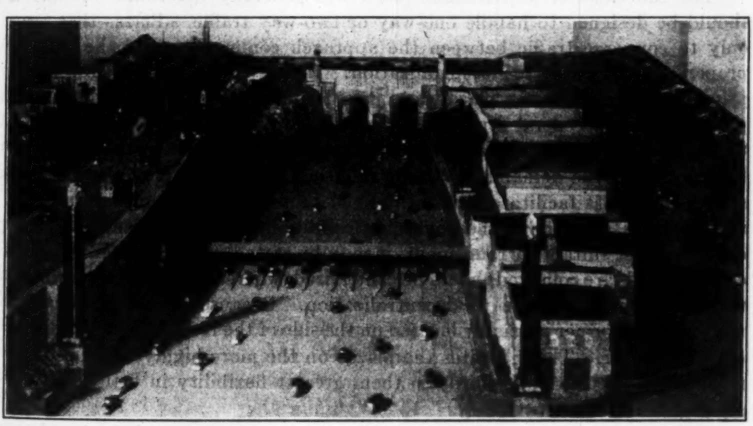

FIG. 7.—A MODEL OF THE NEW JERSEY APPROACH OF THE LINCOLN TUNNEL WHICH WAS USED IN STUDYING THE PLAZA FUNCTIONS OF TOLL COLLECTION AND LANE REDUCTION

New Jersey approach of the Lincoln Tunnel (Fig. 7) has an extensive area, 835 ft in length and 181.5 ft in width at one section, to facilitate the convergence of traffic and the collection of tolls.

Separation of Entrance and Exit Plazas.—Generally the first consideration in plaza design is the arrangement of the entrance and exit facilities. The Holland Tunnel engineers made a forward step in this problem when they decentralized the traffic at both approaches of the tunnel by separating the entrance and exit plazas. This arrangement provides greater access to the tunnel for inbound vehicles, and enables outbound vehicles to disperse more rapidly. Thus far, this method of traffic segregation has been limited to twin-tube tunnels, however, because the separated tubes are more easily adapted to individual plazas at each end than a structure carrying two-way traffic.

The purpose of decentralization is not to keep the entrance and exit plazas apart, but to reduce the overlapping and cross-currents of traffic. Studies of traffic flow at the New York approach of the Lincoln Tunnel demonstrated that unless the two plazas are arranged to minimize this conflict, the congestion in the surrounding area may be as severe as that around a centralized terminal. One approach proposal for the Lincoln Tunnel was rejected when its traffic flow diagram revealed that Tenth Avenue in New York City would have to handle 1,850 vehicles of tunnel traffic at peak hours in addition to its regular volume.

The functions of a particular plaza will generally determine whether it should be designed to handle one-way or two-way traffic. Plazas that serve only to converge traffic between the approach connections and the crossing proper are adaptable to decentralization. However, toll collection is also a primary function of some plazas, and it is desirable from the operating standpoint to have a central arrangement of toll booths. One line of booths for traffic in both directions permits greater flexibility in adjusting the number of booths to suit the demands of traffic in either direction. It is also easier to supervise, and it facilitates the rotation of toll collectors according to traffic requirements. Although the governing conditions in each instance will indicate the most suitable arrangement, it may be stated generally that the plaza on which tolls are collected should be centralized, whereas that which distributes traffic only is adaptable to decentralization. Because of the greater area required, the former is usually located on the side of the river on which property costs are lower. This liberates the plazas on the more highly developed side from toll collection and affords to them greater flexibility in fitting into the street pattern.

Dimensions of Plazas.—By recognizing the requirements of the two functions, lane reduction and toll collection, it is possible to design the plaza for efficient operation economically. The toll collection facilities determine the width of a tapered plaza, and the distance required for the proper convergence of all vehicles on to the bridge or tunnel roadway determines its length.

In like manner, the design requirements of a reservoir plaza may be calculated, although the procedure is generally reversed. Since the available area is fixed by the block to be cleared, it becomes necessary to determine the arrangement of toll booths which will accommodate both functions to mutual advantage. It has been pointed out that the circular arc layout of booths is not suited to the most efficient movement of vehicles. However, this is not an inadequacy of the booth arrangement, but of this type of approach. If these booths are alined at a sufficient radial distance from the focal point on the plaza, the converging lines of traffic have adequate space in which to merge and the disadvantage of cyclic lane movement is lessened. The semicircular booth arrangement is best suited to the reservoir type of approach because it is accessible to vehicles entering from three sides, and it spreads them out along radial lines in preparation to being signaled into the tunnel.

The common acceptance of the tapered plaza on the approaches of large river crossings has brought with it the straight-line array of booths across the collection area. The Delaware River Bridge, George Washington Bridge, Golden Gate Bridge, San Francisco-Oakland Bay Bridge, and Lincoln Tunnel each have the toll facilities alined across one of the plazas (Fig. 7). This enables the toll supervisor to control the flow of traffic through each toll lane and to increase the number of booths accommodating a uni-directional peak.

Provisions for Toll Collection Facilities.—The number of lanes through the toll collection area depends upon the mechanics of collection, that is, upon the number of toll classifications and the type of registering equipment. On the George Washington Bridge, Golden Gate Bridge, and similar modern structures, it takes between 6 sec and 8 sec for each vehicle to pass the booth, which is equivalent to from 450 to 600 vehicles per hr. Since the operating capacity of each lane on the crossing is about 1,500 vehicles per hr, three toll lanes are required for each lane on the bridge or tunnel. As shown in Fig. 7, one toll island, approximately 6 ft wide, can accommodate two toll lanes. If these lanes are 10 ft wide and the requisite number of islands is added, it is necessary to provide at least 39 ft at the collection section for each lane on the crossing in developing the structure to its capacity.

Provisions for Convergence of Lanes.—If the plan of the railroad advance yard is to be used in designing a plaza, it is necessary to determine the distance in which vehicles can converge without slowing down traffic. Relatively little investigation has been made on this phase of vehicular operation although it has an important bearing on the requirements for passing, on highways and the design of accelerating lanes, as well as bridge and tunnel plazas. The operations of a car in moving from one lane to another on a plaza are similar to those in passing another vehicle on a highway; and, to a limited degree, this analogy is of value in determining the design requirements for traffic convergence on plazas.

Adequate sight distance is required in both operations because it is no more desirable to converge with limited visibility than it is to overtake another vehicle on a curve. On a bridge or tunnel plaza, the governing conditions are altered, however, because the unforeseen hazard of a vehicle approaching in the opposite direction is not usually present. Therefore, it is unnecessary to provide any more than the minimum non-passing sight distance for the design speed of the plaza.

The converging maneuver of a vehicle is a reverse curve, in which a side movement of one or more lanes is accomplished while the vehicle is traveling forward. The path of the vehicle is actually a transitional spiral throughout, but it may be assumed to be a reverse circular curve with sufficient accuracy, due to the variation in driving characteristics. If the amount of side thrust caused by centrifugal force which the car occupants can comfortably withstand ($f = 0.15$) is not exceeded, the forward distances traveled in moving laterally across a plaza are as shown in Table 1. Observations of passing on highways indicate that, after the two vehicles are abreast of each other, the overtaking vehicle requires an average of approximately 3 sec in which to cross back into lane, with approximately 1 sec extra for each additional lane that is crossed. Although these observations are only available for vehicles moving over three lanes, a total lateral movement of 30 ft, the results may be projected for wider pavements, at a rate of 1 sec extra for additional lane, to obtain a check on Table 1. On this basis, the required lengths for lateral movement at various speeds are as given in Table 2. These two tables check reasonably well, considering the variation in driving habits; and, in the absence of more indicative studies, they may be used to determine the length of plaza required for various widths. For example, the plaza of a four-lane bridge or tunnel, if developed for capacity operation, requires a minimum of 12 toll lanes which with toll booths aggregate a total width of approximately 160 ft. At a design speed of 40 miles per hr, its length from toll booths to portal should be 500 ft

TABLE 1.—FORWARD DISTANCE TRAVELED IN MOVING LATERALLY ACROSS A PLAZA

Speed, in miles per hour	Radius, in feet ($f = 0.15$)	Forward Distance, in Feet, for a Lateral Movement of:							
		10 ft	20 ft	30 ft	40 ft	50 ft	60 ft	70 ft	80 ft
20	179	84	118	143	164	183	198	213	228
30	402	126	178	218	253	279	305	328	350
40	715	169	238	291	336	375	410	442	471
50	1,117	211	298	365	421	470	515	555	593

TABLE 2.—FORWARD DISTANCE TRAVELED IN MOVING LATERALLY AFTER PASSING

Speed, in miles per hour	Forward Distance, in Feet, for a Lateral Movement of:							
	10 ft	20 ft	30 ft	40 ft	50 ft	60 ft	70 ft	80 ft
Equivalent number of lanes	1	2	3	4	5	6	7	8
Average time, in seconds	3	4	5	6	7	8	9	10
20	88	118	147	176	206	235	265	294
30	132	176	221	265	309	353	397	441
40	176	235	294	353	412	470	529	588
50	221	294	368	441	515	588	662	735

in order to permit the vehicles to move laterally from the outer toll booths to the lanes of the structure. This design speed is considerably greater than that at which the vehicles will operate in traveling from the booths to the portal for, at an average acceleration of 2.5 ft per sec^2, they require 690 ft to attain 40 miles per hr. However, the speed of the off-bound vehicles approaching the toll booths is the governing criterion, and it is well to assume the design speed on the plaza to be the same as that at which vehicles are expected to operate on the crossing proper. The New Jersey plaza of the Lincoln Tunnel has thirteen toll lanes and a total width of 181.5 ft. Its length from the booths to the portal is 591 ft. Similarly, the sixteen toll lanes on the San Francisco-Oakland Bay Bridge plaza are converged to six lanes in a distance of approximately 600 ft.

On plazas constructed solely for toll collection, the convergence distance on each side of the booths should be equal since the movements of convergence and divergence are identical on both sides. This type of diamond-shaped plaza was built on the Oakland approach of the San Francisco-Oakland Bay Bridge (see Fig. 2).

Plazas which only assemble or disperse traffic to connections are generally not as long or as wide as those on which tolls are collected. Since traffic does not have to stop, it is only necessary to provide adequate space in which to secure the proper lane reduction between the connections and the roadway of the structure. These plazas are designed according to the principles of convergence distance, as outlined herein, each application being dependent upon the number and type of connections.

Reservoir Space.—The principal advantage of the reservoir type of approach is its provision of adequate reservoir space to take care of the fluctuating surges of traffic to and from the streets during peak periods. As previously stated, this provision is not necessary if the approach is designed to handle traffic without storing it, since each section is capable of handling all the traffic that passes it without interruption or delay. The storage capacity need only be considered in so far as a reservoir may be necessary due to an irregularity in the flow of traffic or as a safeguard in an emergency.

Certain practical limitations to the uninterrupted flow of vehicles may occur, particularly at points of convergence or divergence. The tendency of drivers to slow down while converging from one lane into another may throttle the traffic behind that point at times. Similarly, an emergency, such as a fire or accident, will slow the velocity of traffic on the plazas, and occasionally will cause it to store there temporarily. Lift-span bridges require a reservoir space, normally, and it is also useful in the classification of slow and fast traffic when high-speed and low-speed lanes are provided on the main structure. In spite of these, however, it is seldom necessary to design the plazas for a predetermined storage capacity, since the space provided for convergence and divergence is a more than adequate reservoir.

Portal Transition.—It takes the average motorist about 5 sec to overcome, completely, the temporary blindness that occurs at the sudden transition from a plaza into a tunnel. In average daylight, the level of light intensity on the plaza is several thousand lumens per square foot, whereas in the tunnel this level is less than 40 lumens per sq ft. The retinal adaptation to this change is a function of the photo-chemical action of the receptor neurons of the eye, which physiologists have found to be dependent not only upon the difference in intensity levels, but, more importantly, upon the suddenness of change in levels.

For this reason, it is essential that the transition between the covered and the open roadway be made as gradual and protracted as possible. The designers of the Lincoln Tunnel have achieved this gradual transition in a measure by flaring the tunnel ceiling upward at the ends of each tube. With the higher opening, more natural light is admitted and the effect is a higher intensity level at these sections, which enables the motorist's eyes to make the retinal adaptation more readily.

The proposed Delaware River Tunnel will also have a flared ceiling at each end, and, in addition, the designers plan to use higher candle-power lights at the portals. This effort at protracting the transition is commendable, but the possibility of producing the reverse effect at night must also be guarded against. On the plaza the illumination may be reduced to only 0.1 lumen per sq ft on a dark night and the sudden change from the well-lighted tunnel will produce the same blindness.

No matter how gradual this transition is, the portal is a critical location. It is necessary, therefore, to make every effort to safeguard the flow of traffic and to minimize the possibility of accidents. In this transition zone which may be considered to extend 100 ft on each side of the portal, any obstructions, changes, or curves are undesirable because the motorist has little opportunity to perceive them sufficiently in advance. The transitions of the Holland and Lincoln tunnels are subject to criticism in this respect because a relatively sharp curve is begun just at the portals in both structures.

Pedestrian Crossings.—Whenever a plaza is to be constructed in an urban district, it is advisable to make a survey of pedestrian movements in the immediate vicinity and to incorporate in the design some means of re-routing this foot traffic if the volume is appreciable. In the highly developed sections which envelop the Holland Tunnel, Delaware River Bridge, and similar approaches, it cannot be expected that the large expanse of plaza area will remain clear of pedestrians unless crossings are provided. Rather than detour 500 to 1,000 ft around the plaza, they prefer to take the chance of "jaywalking" across it. Observations at busy street corners indicate that the percentage of "jaywalking" and irregular pedestrian movements increases with the volume of pedestrian traffic. A small number of people is generally orderly; but as the volume increases, they become increasingly more unwieldy to control. Therefore, the disorder and accident hazard is progressively greater.

Manual control is effective if enough officers are assigned to this duty, but its cost exceeds that of providing a separate pedestrian crossing. It is usually cheaper to install an over- or under-pass for any volume of traffic that will warrant policing. The type of pedestrian crossing depends, of course, on the prevailing conditions at the plaza. One method of subvention is to construct a walkway over the toll booths. Since the booths are generally roofed, it costs only slightly more to lay a sidewalk pavement on them and to construct the stairways at each side of the plaza.

Another situation which causes an inconvenience to motorists and an accident hazard to pedestrians is the stopping of buses at the plaza to load and unload passengers. When a river crossing is opened initially, bus stops are usually prohibited, but quite frequently the volume and character of bus transportation using the crossing has increased to the point where a stop on the plaza is necessary. Ordinarily, it is not possible to forecast the trend in common carrier service sufficiently to be able to design facilities for these vehicles at the time of initial construction. When the volume of bus transportation does become sufficient to warrant a stop at the plaza, however, a bus station should be constructed nearby. This will permit the buses to clear the plaza and still accommodate their passengers, without disrupting the vehicular operations on the plaza.

Architectural Treatment.—The imposing nature of bridge and tunnel plazas demands that due consideration be given to the amenities of these structures and the contiguous properties. Since the period of the Italian Renaissance, the ample spaces at bridge-heads have been treated by architects for formal beautification. Notable present-day examples are the decorative bridge plazas of Pont Neuf and Pont de la Concords over the Seine in Paris, France.

It is wholly desirable to adorn the plazas of a bridge or tunnel so that they will be attractive. The value of such treatment is not only reflected in the pleasant reaction of the driving public, but also in the enhancement of the surroundings.

In this generation, however, the speed of traffic has increased to the point which precludes the opportunity for automobile occupants to appreciate any extraneous adornment. The stress has been on the practical rather than the beautifying features of plazas and the result has been an adaptation of esthetic proportion consistent with their usage. In developing the amenity of a bridge or tunnel plaza, the functions of the plaza should be kept in mind because the complexities inherent in high-speed operation demand the constant vigilance of the motorist. Any extraneous treatment, such as statues, fountains, etc., are distracting and should be avoided or placed out of the driver's view. Numerous architectural designs have been developed in which a pleasing effect is created as a whole without directing attention to any particular part of it, and it is well to retain this type of treatment on bridge and tunnel plazas.

CONCLUSION

For a particular bridge or tunnel, the selection of a type of approach, or the development of approach connections and plazas, which is best suited to traffic demands is a matter of economics and engineering judgment. This paper contains some suggestions of the more significant elements, and their places in the design of bridge and tunnel approaches. The important point to be gathered is that the approaches are the valves which control the capacity and the accessibility of the structure. As it has been stated by Commissioner

Robert Moses, executive officer of the Triborough Bridge, Whitestone Bridge, Henry Hudson Bridge, and Marine Parkway Bridge:[7]

> "It is only in recent times that those advocating new bridges and tunnels, and the public officials responsible for voting money and contracting, have been willing to concede that generous approaches and connections to main arteries are even more important than the crossings themselves, and that there is no justification for building these structures without providing adequate means to get to and from them. The building of approaches as an aftermath is always expensive and frequently impossible, but it has been up to now the rule and not the exception."

ACKNOWLEDGMENT

This paper is abstracted from a thesis presented to the Harvard University Bureau for Street Traffic Research, in 1937, as required by the terms of a research fellowship. The writer is indebted to many professional men engaged in the construction and operation of bridges and tunnels for their assistance, guidance, and criticism. Special acknowledgment is due to Richard L. Steiner and Thomas P. Quilty, Juniors, Am. Soc. C. E., for their valuable assistance.

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[7] *New York Times*, July 9, 1939.

DISCUSSION

Dean G. Edwards,[8] M. Am. Soc. C. E. (by letter).—It is probably true that between 25% and 50% of the total cost of modern vehicular bridges and tunnels is expended on their approaches, but unfortunately this has not always been the case, particularly in times long past. The present inefficient use of some of the older bridges over the East River in New York City is evidence that too little consideration was given to approaches when these bridges were built; or it may be fairer to state that traffic has changed so radically that such approaches as were provided have become obsolete.

Brooklyn Bridge is used for only a fraction of its potential capacity because traffic cannot leave or enter the Manhattan end except by two traffic lanes in each direction, and it is subject to repeated interruptions of serious proportions because of pedestrians on Park Row and because of the light-control system. A costly major operation must now be made if the bridge is to be used to its potential capacity.

The identical conditions do not exist for all the older bridges. The Manhattan and Williamsburgh bridges are more efficient in traffic movement than the Brooklyn Bridge because the approach streets are either wider or more numerous, and there is some opportunity to even the "humps and hollows" of the traffic curve which are caused by the traffic lights.

Among the more troublesome traffic points are the approaches of the Queensborough Bridge. This bridge has five traffic lanes on the main level and two lanes on an upper level. The direction of traffic on the upper level conforms to the direction of the heavier load. It is Manhattan-bound from midnight to noon and Queens-bound from noon to midnight. The upper road was first used in 1931, 22 years after the bridge was opened, and a diversion viaduct was built in Queens so that the upper roadway traffic enters and leaves some blocks distant from the main-level traffic. In Manhattan, the upper roadway turns to the south over 59th Street and exits into either 58th Street or 57th Street through a right of way a little east of Second Avenue.

The main level of the bridge opens into Second Avenue, a light-controlled thoroughfare with frequent interruptions of traffic resulting from the 90-sec light cycle. An exit into 60th Street and an entrance from 59th Street help a little toward the development of bridge capacity, but traffic jams are frequent at peak hours. The lack of proper approaches has not only diminished the traffic flow over the bridge but has had an extremely adverse effect upon the street traffic and property values in the area in proximity to the bridge. Second Avenue is usually closed to north-bound traffic at the bridge (except buses), and instead of the bridge traffic making up the difference between Second Avenue capacity and the normal north-bound traffic, the normal north-bound traffic on Second Avenue is forced to detour. The traffic load of First Avenue, the nearest wide thoroughfare without elevated railroad columns, is about at its ultimate capacity during peak hours, and Second Avenue fails to help by performing its rightful function. The nearby crosstown streets of this vicinity are often filled to capacity, and a blight on property has resulted.

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[8] Cons. Engr., Borough of Manhattan, New York, N. Y.

Plans have been devised for easing the present undesirable situation. Two fundamentally different schemes have been proposed. One provides for depressing the bridge traffic lanes under Second Avenue into a plaza which would occupy the full width of the block between 59th and 60th Streets, nearly to Third Avenue. A new street cut through to 57th Street will permit the traffic to disperse through the cross streets. A study of this plan reveals that the main points in its favor are that it returns Second Avenue to normal use as a north-south artery and that a broad plaza is provided as a reservoir for equalizing the interruptions of flow caused by the stop lights. The objections to this plan are: (1) Its high cost; (2) the fact that much of the traffic over the bridge will desire to reach the East River Drive, and this routing would bring it to the wrong side of Second Avenue; and (3) the moving of the bridge outlet to the west side of Second Avenue would not in itself solve the problem at all, but would merely move the congestion a half a block toward the west.

The alternate solution entails a widening of Second Avenue from 100 ft (between building lines) to 175 ft for about seven blocks, and passing the normal traffic of Second Avenue by the bridge entrance in a four-lane tunnel with staggered entrances and exits. The widened avenue would provide enough lanes for reservoir space for equalizing the stops caused by the light system, and the bridge traffic, at full capacity, would not exceed the capability of the approach streets to absorb it. The elevated railroad on Second Avenue complicates the problem, but the proposed early demolition of this structure may remove this obstacle. In any case, this problem must be considered in all its aspects, including the entrances and exits to both the north-bound and south-bound roadways of the East River Drive.

The East River Drive from 93d Street to 125th Street was constructed as an approach to the Triborough Bridge. It will become a part of an express highway without light control, having three lanes of traffic (11 ft per lane) in each direction, and a limited number of entrances and exits.

The importance of estimating bridge capacity in terms of the capacity of approach streets to absorb the traffic cannot be overemphasized.

There is considerable doubt as to the usefulness of traffic circles as regulators of traffic. In outlying districts they seem to be fairly satisfactory, but they are a nuisance to through traffic which must slow down to a greatly reduced speed. The objection to them in Manhattan is not that they are uneconomical because of the high cost of real estate but that they are unsatisfactory under heavy traffic conditions. Traffic circles, under heavy traffic, induce many minor accidents, slow traffic unreasonably, are difficult to operate, and do not obviate the use of lights. The experience in Washington, D. C., where Massachusetts Avenue is depressed under Thomas Circle, is rather conclusive. Any one who has often driven a car around Du Pont Circle during peak load hours cannot retain enthusiasm for traffic circles in large cities.

The examples of present problems created in Manhattan because of failure to foresee the vast volume of traffic, which has developed, and to plan accordingly are cited to show the necessity for the substantial part of bridge costs which Mr. Curtin mentions as properly expended on approaches. Earlier realization of this need and proper planning for adequate approaches would have simplified the present problem of reconstruction immensely. Even the admirably designed approach system of the Lincoln Tunnel has created traffic problems in Manhattan, because of bus concentration outside the limits of Dyer Avenue. It is inevitable, perhaps, that capacity traffic of tunnels and bridges will always create traffic problems in densely populated areas outside the distribution plazas.

JOHN W. BERETTA,[9] ESQ. (by letter).—Results of careful study and research are evidenced throughout Mr. Curtin's comprehensive paper. He is to be congratulated upon a concise and compact treatment of a subject which, because of its diversity and many possible ramifications, might have extended to an almost impossible length. He has indeed set a good example by controlling his subject in as efficient a manner as he suggests for the disposal and expedition of traffic in the approaches to bridges and tunnels.

One quite logical conclusion that may be drawn from this paper is the fact that only quite large and monumental bridges and tunnel developments have received adequate and serious design effort on approaches. In other words, the amount of approach planning for a particular structure is roughly proportional to the cost of the main structure, its size, and its importance as a link in the highway system it serves. There is no doubt that even the smaller and less important crossings could be improved materially if more attention were given to approach design study, but in the past this has been consistently overlooked or purposely disregarded due to limited financing.

Another significant fact emphasized in this paper is that all of the structures discussed as illustrative examples are of rather recent construction. This would lead to either of two natural conclusions: First, that older structures of comparable magnitude and importance did not have comparable approach problems because of less density and velocity of traffic; or, second, that the importance of approach problems was overlooked, either purposely or because a lack of study and knowledge prevented recognition of their importance. A study of some crossings older than those discussed by Mr. Curtin undoubtedly will reveal that both conclusions may be valid although they may vary in individual cases. This may serve to some extent to explain his statement that "relatively little of the diligent research and analysis * * * has been made available to the engineering profession."

The statement regarding the paramount importance of the kinetics of vehicles is very significant in that it tends to bring to mind the fact that older structures did not have comparable approach problems. Also it tends to prove that bridge and tunnel approaches are highly susceptible to obsolescence due to changes in type, density, and velocity of traffic which, in many cases, can become serious before the obsolescence of the structure itself, regardless of the cause. There are many structures in use today which, even if they were actually built for traffic of "horse and buggy" velocity, could accommodate modern high-speed traffic if their approaches were amplified and modernized properly. Their usefulness, based on Mr. Curtin's data, could be increased from 50% to 100%, and such reconstruction might avert the necessity of constructing an entirely new facility.

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[9] Cons. Engr., San Antonio, Tex., and President of the Am. Toll Bridge Assoc.

As surely as adequate foundations are necessary for the stability, load capacity, life, and safety of any structure, so are adequate approaches necessary for the utility, life, economic value, traffic capacity, convenience, and traffic safety of a structure. This analogy is the more significant when one ascertains that the percentage of cost of the substructures of a crossing are more than likely to be about the same as the percentage of cost of the approaches. It has been difficult, in the past, to convince laymen and even some engineers of these facts, but the evident progress, as outlined by Mr. Curtin, is most encouraging and certainly is a trend in the right direction.

During the past few years the toll bridge industry in general has been under rather withering fire from governmental highway agencies seeking complete elimination of the toll bridge as a public servant, so that the traveling public can be provided with "free" bridges. In this campaign to provide these "free" bridges (so-called because no direct toll is paid although indirectly a toll is exacted of every taxpayer whether or not he uses the bridge), arguments in favor of the "free" bridges have been offered which have been confusing and misleading to the general traveling public. Because of their approaches, which due to the rapid change of traffic volume and speed are becoming inadequate, many crossings adequate in all other respects have been condemned unjustly and a loud clamor has been raised for superseding or duplicating facilities when only approach improvements and extensions are needed for the existing facility.

To the toll bridge industry, therefore, the general problems of bridge approaches are increasingly important. Not only must new structures be provided with properly designed approaches, but old structures, if they are to survive these—for the most part unwarranted—attacks, must be rehabilitated through rational modernization and amplification of approaches. Toll bridges have been hailed through the ages by economists as the fairest and most equitable crossings because they are paid for by the actual users of the facility and not by long-suffering taxpayers who may never even see them.

Despite the fact that it is not generally known, it is true that the toll bridge, as well as the toll road, type of facility is again growing in use and acceptance by the public. The toll bridge industry originally was founded only on the basis of the investment of private capital. It is only in comparatively recent years that public and quasi-public agencies have constructed toll facilities financed through the issuance of self-liquidating securities. The privately financed toll bridges, therefore, are pioneering ventures and as such are entitled to recognition for the performance of a public service in a time of necessity. It is significant that in Mr. Curtin's paper most of the specific examples are crossings of the toll type, paid for by revenues instead of taxes.

Of the various groups and types of existing bridges which need modernization of approaches, none presents more acute problems than international structures, particularly those along the Mexican border. Here, however, the complicated problems of traffic capacity and distribution come at the exit instead of at the entrance to the crossing. Each international crossing is provided with inspection facilities for four governmental agencies—namely, the Public Health Service, the Bureau of Customs, the Department of Agriculture, and the Immigration Department. These facilities are in duplicate, with the American and equivalent foreign agencies each occupying one approach to the structure. The time element involved in the inspections by these agencies enormously complicates the handling of traffic, especially when it is necessary to revise tourist baggage and conduct the other business of a port of entry.

The most important and pressing problems developed by Mr. Curtin are those of entrance; and his exit problems are chiefly questions of geographical distribution and the clearing of off-bound vehicles through cross traffic. In connection with an international crossing the problem somewhat reverses and the chief concern is one of exit. In these cases, however, off-bound traffic must be sorted into three major groups—local, freight, and tourist, with each group requiring a distinct handling. The consideration of geographical distribution and cross traffic clearance at the exit is of minor importance when compared to the other problems.

In the days before the introduction of through trunk highway systems all of the traffic on these international bridges was purely local in character with little complication or delay involved in governmental inspections. At that time all freight and tourists with baggage were carried on the railroads. Now, however, the situation is changing to completely outmode the existing approaches, which, with few exceptions, are of the "direct street or highway extension" type as described by Mr. Curtin. With privately-owned tourist automobiles traveling far afield and with an enormous tonnage of freight being carried by motor trucks, acute conditions are arising from the long queues of off-bound vehicles extending back on the structure proper while awaiting the unwaveringly complete inspections. On-bound traffic at each end of the structure can be speeded through utilization of the known methods suggested by Mr. Curtin, but the expedition of off-bound traffic is a problem which is still under process of practical solution. Probably this solution will be found in the construction of reservoirs where cars detained for inspection may be parked out of the way of local traffic not requiring luggage revision.

The first tangible recognition of the necessity for approach improvements to an international crossing has come from the United States Congress. In 1939 this body appropriated $290,000 for the construction of a Port Terminal Building at the existing privately owned and operated bridge at Laredo, Tex. This improvement will be of great benefit in accommodating the increasing port business at this point, which is the greatest of any Mexican border port. Experiences gained in this development by the Federal Works Agency will be highly valuable when consideration of other similar problems is undertaken.

The International Highway Bridge between Laredo, Tex., and Nuevo Laredo, Mexico, is a large mounumental concrete arch structure which forms an important link in the Inter-American Highway extending from Canada almost to Central America and which, ultimately, will extend to South America. With the exception of the exit facilities (outmoded because of a change in the type of traffic handled), the bridge itself is considered adequate for all traffic requirements. With the completion of the new governmental port terminal facilities, which will modernize the approach completely, the bridge again can function properly to accommodate the full capacity of its roadway. This is a clear-cut example of a toll facility that can be restored to full usefulness by approach amplification without the need of an unnecessary duplication of the facility.

At first the problem at Laredo is one of segregation or separation of various types of traffic at the exit, because each required different types of inspection. The largest volume consists of local passenger-car and empty-truck traffic whch normally can flow with only casual inspection and a slight delay equivalent to that necessary for the payment of toll at the entrance. The next to merit consideration is the very troublesome loaded-truck traffic which is subject to inspection delays of from a few minutes to several days in extreme cases. The most serious and troublesome problem is experienced with passenger cars laden with tourists, varied baggage, and huge quantities of merchandise souvenirs, the problem being complicated further when the tourists are aliens. Added to this conglomeration are the additional problems of loading and unloading platforms, office facilities, rest rooms, waiting rooms, detention spaces, fumigation and disinfecting facilities, warehouse storage, and many other related factors.

In the solution of the problem many ideas are being borrowed from railroad practice which long has been concerned with segregation and rational handling of varied types of traffic on a restricted basis of entrance and exit. Also ideas have been considered from the practice of water-borne traffic handling at ports of entry along the seacoast. Together with these, however, ideas are also adapted from the latest bridge approach practice as described by Mr. Curtin, and partake of the essential features of nearly all of the types he describes. The local traffic travels in the same manner as it would on a direct street or highway extention with some tapered plaza and feeder connection modification. The other types of traffic must be accommodated by a series of segregating reservoirs designed to hold a maximum of each type. The whole must be correlated with the other features of a port-of-entry terminal as described previously.

It can be seen, therefore, that the bridge-approach problems of an international crossing are much more complicated than even the larger interstate or intrastate crossings. With the rapid increase of international highway traffic, particularly to Latin America, the problem is really an acute one, and a satisfactory solution must be found if this traffic is to be handled adequately. The studies at Laredo undoubtedly will afford a satisfactory solution of that particular problem and will point the way to the final answer to the special problems of other international crossing locations, either Canadian or Latin American.

George Hartley,[10] Esq. (by letter).—Problems related to the proper design of bridge and tunnel approaches are increasing rapidly with the present extensive use of toll charges on highways in the United States. Typical construction is exemplified in the present trend toward toll express highways and parkways as well as the steady increase in the number of toll bridges and tunnels. In his excellent paper, Mr. Curtin has summarized many of the conditions and problems related to the present major toll bridge and tunnel construction. Some attention should be given to the great percentage of toll structures on which the toll collection requirements are for only a small volume of traffic which, in general, does not approach the traffic volume attained by the structures analyzed by Mr. Curtin.

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[10] Engr. with Robinson and Steinman, Cons. Engrs., New York, N. Y.

An obsolete type of toll booth is shown in Fig. 8, at which two-way toll is levied by a single collector. A number of these structures are still in existence in rural low-traffic areas and are "left-overs" from the day of the horse and buggy. The toll gate placed across the highway required all vehicles to stop; and the barrier was raised after the toll had been collected. In bad weather the toll collector is exposed to the elements, and at all times there is the danger, during heavy traffic periods, of being run down by vehicles when the highway is crossed to collect tolls.

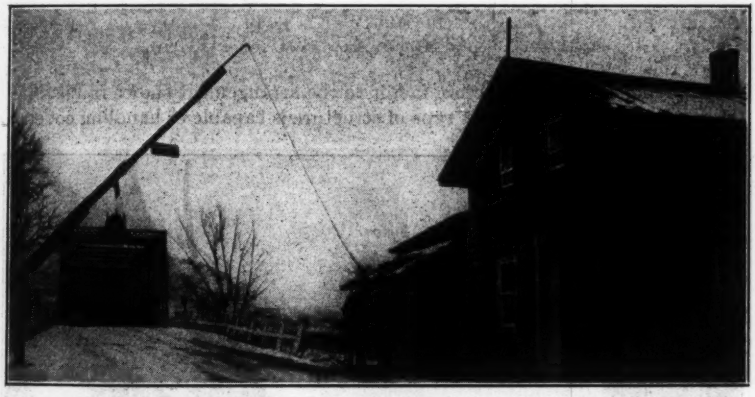
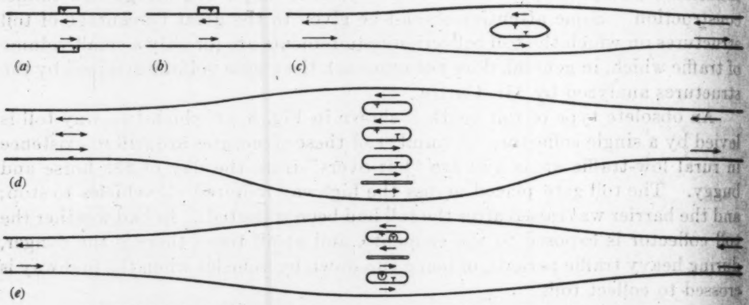


FIG. 8.—TOLL HOUSE OF THE SMITHBORO BRIDGE COMPANY, SMITHBORO, N. Y.

This discussion centers about the relative traffic-volume capacity of the various toll plaza arrangements considered. The basic assumptions upon which these evaluations were made are as follows: A collection time of 6 sec is assumed as a constant factor when toll collection is made by the driver of the vehicle out of the left-hand or driver's window; when the driver is required to reach across the front seat to pay toll, a collection time of 8 sec has been assumed. These assumed applications of time are for use in relative traffic-capacity considerations and are based on the 6-sec to 8-sec collection periods given by Mr. Curtin as representative of the conditions at the George Washington Bridge and the Golden Gate Bridge. The 2-sec differential is substantiated in a measure by Glenn B. Woodruff, M. Am. Soc. C. E., in his observation of traffic conditions that exist at the toll house of the Trans-bay Bridge (correspondence with the writer). At this crossing traffic that pays toll from the driver's side of the vehicle moves freely, whereas in the adjacent lane, which requires the driver to reach across the front seat in paying toll, an appreciable lag has been noted.



(a) 450 Vehicles per Lane per Hr
(b) 225 to 450 Vehicles per Hr
(c) 600 Vehicles per Lane per Hr
(d) 1,800 Vehicles per Lane per Hr
(e) 1,050 Vehicles per Lane per Hr

FIG. 9.—TOLL-BOOTH LAYOUTS, SHOWING MAXIMUM CAPACITIES

Applying the 8-sec collection factor to the arrangement shown in Fig. 9(*b*), the maximum traffic that this type of structure is capable of handling covers a range of from 225 to 450 vehicles per hr (see Fig. 9(*b*)). Even the value of 225 vehicles per hr may be excessive as the collector must go to the center of the highway in order to collect toll from traffic in the far lane which, in other types,

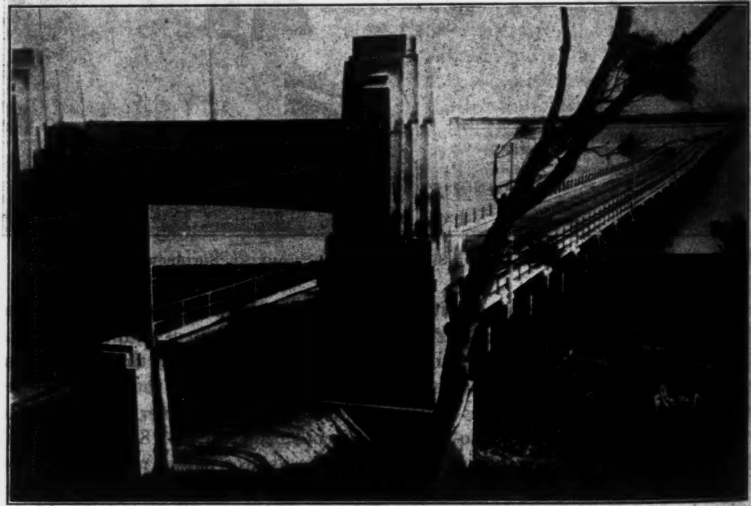


FIG. 10.—TOLL HOUSE FOR THE HORNIBROOK BRIDGE, AUSTRALIA

would require only a 6-sec collection time. The time to cross the highway may be considerably more than 2 sec.

With the development of the motor vehicle, and the subsequent increase in highway traffic, another toll house was added as illustrated diagrammatically

FIG. 11.—TOLL HOUSE OF THE WALNUT STREET BRIDGE, HARRISBURG, PA.

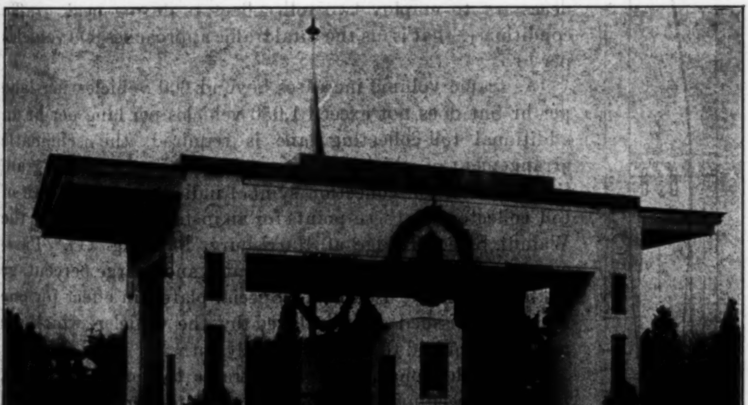


FIG. 12.—TOLL HOUSE OF THE MOUNT HOPE BRIDGE, RHODE ISLAND

in Fig. 9(*a*). Thus tolls were collected separately for each traffic lane and the maximum traffic that could be handled was 450 vehicles per lane per hr. In this type the collection of toll is not convenient as the operator is still required to reach across the front seat in order to pay toll. A typical installation of this type is shown in Fig. 10. Traffic keeps to the left. Drivers seated on the left side of the vehicle can pay toll conveniently; and a maximum traffic capacity of 600 vehicles per lane per hr is possible. As most vehicles in Australia are operated with the driver on the right side of the vehicle there is little or no increase in traffic capacity over the arrangement shown in Fig. 9(*a*). The total capacity range under operating conditions shown in Fig. 9(*a*) will range from 225 vehicles per hr with one toll collector on duty to a maximum traffic capacity of 900 vehicles per hr when two toll collectors are on duty.

In Fig. 9(*a*) two toll collectors would be required for normal operating conditions, and in localities where traffic volume is small an added expense is incurred. This condition may be remedied by placing the toll booth in the center of the highway, as shown in Fig. 9(*c*), so that the drivers in both lanes are placed for convenient toll collection with a resulting traffic capacity of 600 vehicles per lane per hr when two toll collectors are operating. As stated, this form is the best for low traffic conditions as one collector is able to collect tolls at a rate that approaches 600 vehicles per hr, and it is only necessary to employ two toll collectors during peak traffic conditions—that is, as the total traffic approaches 600 vehicles per hr.

As traffic volume increases beyond 600 vehicles per lane per hr but does not exceed 1,050 vehicles per lane per hr an additional toll-collecting lane is required, the schematic arrangement of which is shown in Fig. 9(*e*). In this diagram the encircled toll collection symbol indicates the absence of toll collectors at these points for an installation made by the Walnut Street Bridge at Harrisburg, Pa. (Fig. 11). Passengers are allowed commutation rates and a large percentage of the total traffic use them. Special plates, in effect for one month, are issued and displayed by the vehicles, which are then not required to stop at the toll booth.

With additional increases in traffic to more than 1,050, but not exceeding 1,650, vehicles per lane per hr, additional toll collection facilities must be provided; and, with the type illustrated in Fig. 9(*e*), an additional toll collecting lane must be provided for every alternate increase in traffic of 450 and 600 vehicles per lane per hr. Under the condition of maximum design of 1,650 vehicles per lane per hr the layout in Fig. 9(*e*) is extended to accommodate an additional toll booth

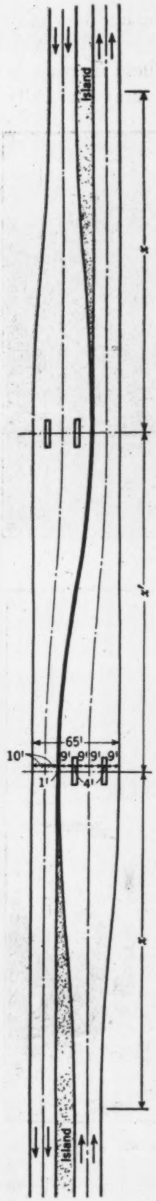


FIG. 13.—SUGGESTED ARRANGEMENT OF TOLL BOOTHS TO ALLOW GREATEST FREEDOM TO TRAFFIC MOVEMENT

which is placed on the center line of the highway so that two lines of traffic traveling in opposite directions are separated for toll collection. This condition is illustrated by Mr. Curtin in Fig. 2, for the installation at the Transbay Bridge. Under this condition, there are three toll-collecting lanes capable of handling 1,650 vehicles per lane per hr, and the author's statement that three toll-collecting lanes are required to pass 1,500 vehicles per lane per hr is substantiated. At the annual meeting of the Highway Division in New York, N. Y., on January 18, 1940, Guy Kelcey, M. Am. Soc. C. E., quoted actual highway traffic conditions in which traffic volume was approximately 1,800 vehicles per lane per hr at an estimated speed of 35 miles per hr. Sigvald Johannesson, M. Am. Soc. C. E., has given an equation for maximum density of traffic which, at vehicle speeds of 35 miles per hr and 40 miles per hr, gives maximum values of 1,812 and 1,869 vehicles per lane per hr.[11] These values are in excess of the author's design value of 1,500 vehicles per lane per hr, and four toll lanes would be required in the extension of the design in Fig. 9(*e*) making possible the collection of tolls for traffic as great as 2,100 vehicles per lane per hr. To pass the foregoing maximum values of traffic without resorting to the use of additional lanes the design given in Fig. 9(*d*) may be adopted which, on the basis of the previous computations, is capable of passing a volume of 1,800 vehicles per lane per hr in three lanes. This theoretical value may be reduced slightly as there is an increase in the horizontal travel of the vehicles across the plaza. An actual installation of this type is shown in Fig. 12 at the Mount Hope Bridge in Rhode Island. In this installation every possible condition of toll collection with respect to varying volume can be accommodated. Fig. 9(*d*) represents the schematic arrangement of this toll booth. The bottom lane, which indicates travel in both directions, was built to accommodate vehicles having unusual clearance requirements.

In closing the writer would like to call attention to Fig. 13 which shows an arrangement for the construction of toll booth plazas that has unusual merit. The toll booths are separated as well as staggered and the diagram illustrates the application to modern express highways where a central parkway strip is used. This solves the problem with regard to maximum-flow capacity in which particular stress is placed on the location of the entrance to the plaza with the criterion that no abrupt changes can occur in the direction of the traffic flow. As shown in Fig. 13, the entrance to the plaza is centered about the toll booths and under low-traffic conditions traffic keeps to the right in approaching the booths. After toll has been collected the driver of the vehicle is not required to travel transversely as he is in line with the continuation of the highway and drives straight ahead. Many advantages are obtained with this design: The horizontal travel of vehicles is reduced to a minimum; for parkway construction the toll booth plaza is kept within the roadway width; the right-of-way purchase is reduced; and, for the condition illustrated, no additional right of way is required. In areas of high realty costs this may result in large savings. The arrangement accommodates traffic volumes as great as 1,050 vehicles per lane per hr; and, with increase in capacity, construction will extend on to the shoulders. For the design in Fig. 9(*e*), peak traffic conditions during the day, with practically all the travel in one direction, may be accommodated by utilizing the toll booths. The latter normally would be used for traffic traveling in the opposite direction, and this is not possible with the design in Fig. 13 as there is an actual segregation of traffic.

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[11] "Highway Economics," by Sigvald Johannesson, 1st Ed., McGraw-Hill Book Co., Inc., New York and London, 1931, p. 90.

GEORGE H. HERROLD,[12] M. AM. SOC. C. E. (by letter).—Smooth uninterrupted traffic flow is the ultimate goal on all traffic routes. This is being attained by slow degrees. A valuable contribution is the special research, by Mr. Curtin, of the studies that have been made of the vehicular approaches of some of the outstanding tunnels and bridges of large carrying capacity built in recent years. In fact, it is a treatise on the subject because it brings together knowledge by description that heretofore had to be looked for under many sources. The idea of treating bridge and tunnel highway approaches as a subject separate and distinct from general highway construction is to be commended because these approaches have their own distinct problems. The solution of an approach problem must stand forever, either impeding or increasing the efficiency of the bridge or tunnel.

Traffic fatalities cannot be stopped by legislation or law enforcement or committee meetings, although it is through these mediums that a very valuable educational program may be conducted, the ultimate result of which will be new engineering devices for building safety into highways. To insure smooth uninterrupted traffic flow, it is necessary to build into approaches some necessary features that will make it impossible for the careless driver to make abrupt changes in direction or other unexpected or irregular movements, and that is the principle that has been followed more or less effectively in the approaches studied.

Approaches to confined roadways such as bridges and tunnels should not, in any way, limit the capacity of the structure which, because it has no cross-traffic in its own length, has a capacity of some 1,500 vehicles per lane per hr. Signal installations on thoroughfares at the ends of such structures, however, are likely to cause great trouble as the carrying capacity of signalized thoroughfares is only 600 to 800 vehicles per lane per hr. It is to be noted that many of the structures that have been studied are of the self-liquidating type, indicating, perhaps, that more attention has been given to approaches where tolls are charged than to approaches to tunnels and bridges which have been paid for through taxes. Although the author deals with large structures, the same principle applies to smaller ones and many of these can have their life's usefulness extended and improved by using the data collected by the author.

Mr. Curtin mentions the studies made by Mr. Evans,[3] in which he made use of certain principles laid down by railroads. It is interesting to note this gradual turn toward railroading principles and the use of tapering converging lanes. A railroad train travels in a lane on rails and can change that lane only at stated intervals; and this may be the ultimate principle that will control the handling of vehicular traffic.

The author makes no mention of city planning or the necessity of a complete coordination of movements of vehicles to, and from, bridge and tunnel approaches and the thoroughfare plan of the city. The influence of the Regional Plan of New York, however, can be found in the New York City entrance to the Holland Tunnel and in the New Jersey and New York approaches of the Lincoln Tunnel. Its influence can be found in other approaches, although the approach itself may not have been perfect because, as the author states, vested interests and expensive demolitions may prevent following the plan.

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[12] Planning Engr., City Planning Board, St. Paul, Minn.

PARK H. MARTIN,[13] **ESQ.** (by letter).—The subject of bridge and tunnel approaches has been covered ably in this paper. The fact that 25% to 50%, and sometimes more, of the cost of modern bridge and tunnel projects is in the approaches, is well established. With this premise accepted, it should follow that in selecting the site for the project, the engineer should consider the topography, in relation to approach planning, with the same care as he considers traffic volumes, traffic collection, and distribution.

Some sites may offer advantages of one kind over advantages of another kind at another site. The topography of one site may suit one type of approach and not be fitted for another type. It seems pertinent to emphasize the fact that, in selecting the site, the engineer should have in mind all the vital elements affecting the project—type of structure, foundations, clearances, type of approach, traffic, property damage, and effect of the structure on the district. The location selected for a non-toll project may be entirely different from one selected for a toll project.

The major part of Mr. Curtin's paper is devoted to the type of project requiring toll facilities. The non-toll, direct street and highway approach receives much less consideration than the reservoir and plaza toll types, although the direct street and highway approach types are far more numerous. The economies involved in the problem of the type of approach are all on the side of the direct highway connection. The economics of the entire subject receives only passing mention in the paper.

Undoubtedly it is true that the direct or highway extension approach lends itself more readily to rural and suburban areas for two reasons—lesser traffic volumes, and costs. If immediate needs are satisfied by the direct approach and if estimated future requirements indicate the need later of a traffic separation device at certain points, it would seem advisable to acquire sufficient right of way in the first instance for use in the ultimate.

The point of weakness in the direct approach is usually at the intersecting main highways that may be feeders to the bridge or tunnel. This fault may be corrected by use of traffic-separation devices or traffic circles. A simple traffic-separation device at an important arterial route, as the beginning of a street extension approach, may be all that is required to make the approach functionally adequate.

The theory advanced by Mr. Curtin that the use of traffic circles is limited to suburban and rural areas, in connection with approach planning, because of the area required and cost, is not borne out in the writer's experience. A type of grade separation that might perform, as effectively, the work of a traffic circle will require almost as much, or more, area as the circle, with a greater physical cost. This is particularly true when the intersecting highways are in a flat area. The traffic circle will also have advantages if there are more than two highways to be accommodated at the same intersection.

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[13] Planning Engr., Allegheny County Planning Commission, Pittsburgh, Pa.

The final decision as to the type of device to be used must be based on the functional adequacy of the device, with due consideration to the cost. On a non-toll project, the approach has only one function—to provide adequate access to the bridge or tunnel. On a toll project there is the added function of toll-collection facilities, which must be ample to permit a reasonably free movement of traffic. In either case, the estimated traffic volumes should, in a large measure, decide the type of approach. On a toll structure, storage capacity from the collection booths to the first cross or intersecting highway must be ample, if a separation device is not used. If the distance is too short for storage to accommodate peak traffic volumes with the ordinary number of traffic lanes, then additional lanes in the form of a tapered plaza should be considered. By extending the additional lane or lanes to the intersection, in a case of no separation device, the capacity of the intersection by light control can be increased greatly.

The tapered plaza at the New York City end of the Lincoln Tunnel functions as a direct approach. The wide entrance collects the traffic, then by means of converging traffic lanes, channelizes the traffic into the two lanes of the tunnel. This principle of additional traffic lanes at intersections has been used in highway construction. An interesting example of this type has been built in Lake County, Indiana, where U. S. Route 30, the Lincoln Highway—a 40-ft highway—crosses U. S. Route 41—another 40-ft highway.[14]

The diamond-shaped toll plaza of the San Francisco-Oakland Bay Bridge, as shown in Fig. 2, is located on a form of a direct highway approach. The only justification for the diamond-shaped plaza is to accommodate toll facilities.

The tapered plaza is a close relative of the square or rectangular plaza. In the writer's opinion, both have their advantages, considering street patterns and traffic volumes from various feeders. Inherently they both work on the same principle of collection and storage from feeder connections.

One of the serious problems affecting either type is unbalanced feeder loads, this problem being particularly serious where toll collection is required.

The reservoir approach is a street-extension approach with a wide throat in the form of a rectangle or square, acting as a traffic collection well, with large storage capacity. Its use belongs in urban areas with heavy traffic volumes and may well be applied to both bridges and tunnels. The shape and size will be governed by street patterns and traffic volumes.

Mr. Curtin illustrates certain weaknesses in the operation of the reservoir plaza at the New York end of the Holland Tubes. With the limited information given, as to traffic volumes, it would seem as if the difficulty is nothing inherent in the reservoir type of approach, but rather a volume of traffic at the peak hour that exceeds the capacity of either the collection booths or the tunnel. Again it may be that traffic density on one of the approach streets may be greater than the collection booths on that side of the plaza can accommodate.

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[14] *Concrete Highways*, September–October, 1938.

The author's further criticism of the reservoir plaza, as being ill-adapted for use as storage space, does not seem to be warranted. This may be no inherent fault of the rectangular or square plaza, but rather in the design of the exits or throat from the plaza into the tunnel.

The use of transition lanes of sufficient length from the plaza to the tunnel should correct this condition. In support of this solution, the writer would refer to the following statement in the paper (see heading, "Definitions: Reservoir Approaches"): "In addition, the twelve radial lines of cars must narrow into two lanes in a relatively short space."

The author criticizes the use of the rectangular or square plaza as being uneconomical in a business or industrial area, due to inability to use all of the space in the plaza because of the shape. The writer does not agree with the author on this point. If the device, whether it is a rectangular, square, or tapered plaza, whether it is a direct street or highway approach, with or without traffic separation devices, functions adequately for the purpose designed, then it should not be questioned. Any other device or type of approach which will not perform the function for which designed, with or without unused space, has no right to be considered.

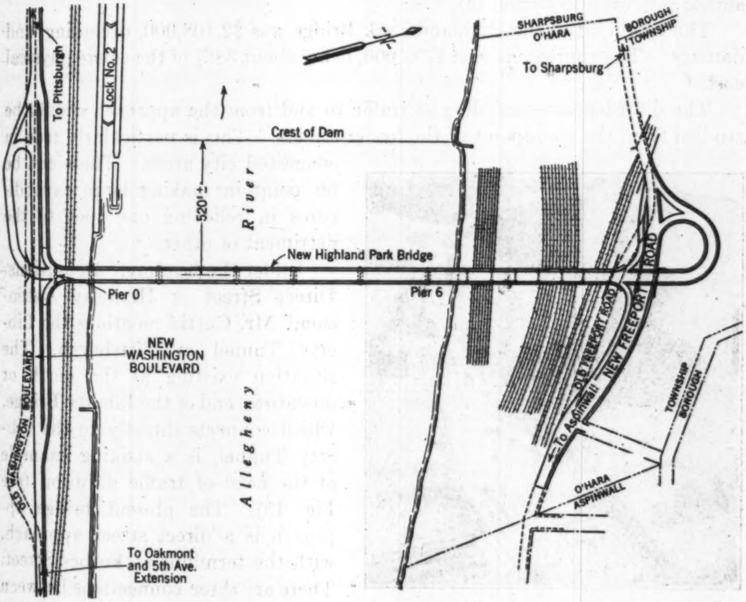
To Pittsburgh
Lock No. 2
Crest of Dam
520'±
River
Allegheny
New Highland Park Bridge
Pier 0
Pier 6
NEW WASHINGTON BOULEVARD
OLD WASHINGTON BOULEVARD
To Oakmont and 5th Ave. Extension
SHARPSBURG
O'HARA
BOROUGH
TOWNSHIP
To Sharpsburg
ROAD
OLD FREEPORT ROAD
NEW FREEPORT
To Aspinwall
TOWNSHIP
BOROUGH
O'HARA
ASPINWALL

FIG. 14.—HIGHLAND PARK BRIDGE AND CONNECTIONS, ALLEGHENY COUNTY, PENNSYLVANIA

For a non-toll bridge there is a type of approach that should be of interest to engineers. A particular case, with which the writer has been associated, involves the Highland Park Bridge across the Allegheny River, with a main highway at each end, both at right angles to the bridge, in what is rather a confined area due to physical conditions. The bridge, a non-toll structure, shown in Fig. 14, has been constructed by Allegheny County, Pennsylvania. Both highways are four-lane arterial routes leading to and from the City of Pittsburgh, Pa., each with traffic volumes in summer of more than 5,000 cars per day. The bridge is built at an elevation to pass over the highways, with approach treatment the same at each end. After passing over the highway, the approaches curve in general through 90°. Then they run parallel to the main highway until the approach grade is common with the grade of the main highway. The approach is finally curved to an intersection with the main highway. The approaches provide for on and off movements from the bridge, with one movement on the highway, with no crossing of traffic. At a point just before the bridge crosses over the main highways, ramps are constructed to the right and left from the bridge to the highway, thus providing access to and from the bridge from the opposite traffic lanes on the main route. The bridge was opened to traffic in June, 1939, and there have been no delays or traffic tie-ups at any time.

The principle of this separation is being used on highway construction and is well adapted to bridge approaches of this type. A striking example of its use on highway construction is on the Pennsylvania Turnpike mentioned by the author following Equation (4).

The total cost of the Highland Park Bridge was $2,108,000, excluding land damage. The approaches cost $793,000, being about 38% of the entire physical cost.

The diffusion or assembling of traffic to and from the approach should be studied from the standpoint of the feeder streets. This is particularly true in congested city areas. There can be no point in making large expenditures in relieving one spot to the detriment of others.

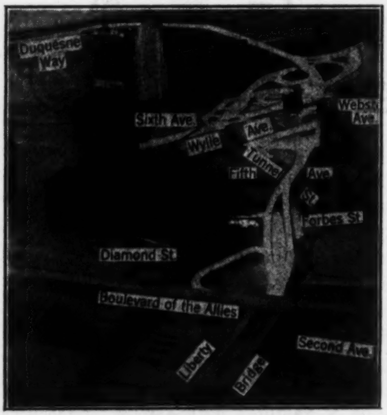

Fig. 15.—Model Crosstown Highway, Pittsburgh, Pa.

Under the heading, "Definitions: Direct Street or Highway Extension," Mr. Curtin mentions the Liberty Tunnel at Pittsburgh. The situation existing at the north or downtown end of the Liberty Bridge, which connects directly to the Liberty Tunnel, is a striking example of the need of traffic diffusion (see Fig. 15). The present bridge approach is a direct street approach, with the terminus at Forbes Street. There are three connections between Forbes Street and the bridge proper. Traffic using these three connections does not represent more than 25% of the bridge traffic. The remaining 75% concentrates at the Forbes Street and bridge approach intersection and 40% of the total bridge traffic has no need or desire to find its way through various streets to and from the Forbes Street intersection. What is needed and planned is a separation of traffic at Forbes Street by means of a crosstown highway, extending to the north edge of the Triangle, separated from all cross streets by means of overpasses and underpasses. The plan, so arranged to allow sufficient points of distribution and access, warrants the belief that it is functionally adequate without superimposing excessive traffic loads at any one point. A complete origin and destination survey of traffic using the Liberty Bridge was necessary to arrive at an intelligent analysis of the problem. The estimated cost of the Crosstown Highway is $7,000,000.

This cost, with the cost of grade separations planned at both ends of the Liberty Tube, will show a total expenditure of $10,000,000 to correct the conditions. The total cost of the Liberty Bridge and Tunnel was $13,000,000. Thus, with the additional contemplated expenditure of $10,000,000, there is, again, more the 40% of the cost of the entire project in the approaches and connections.

Mr. Curtin recommends that in regard to approaches, serious consideration be given to the steady-flow system as developed by Fritz Malcher.[8] The use of elongated separating islands 10 ft to 40 ft wide is recommended. Without any thought of criticizing the general theory of the steady-flow system, the writer will severely criticize the use of any island as part of a steady-flow system that does not provide for a minimum turning radius of 30 ft. The minimum width for the island should be 50 ft. Part of the principle involved in the steady-flow system is a form of a U-turn. Any form of a U-turn is subject to some criticism, and with an island 10 ft wide the U-turn is fatal to the free-flow system.

Use of the freeway or express highway is commended as being worthy of the utmost consideration. Physical conditions and cost should be the only factors against its use. It represents the most advanced thought on traffic movement.

Design of Approach Connections.—Under the heading, "Approach Connections: Design of Approach Connections," Mr. Curtin has discussed various factors that should be considered, such as design speed, sight distance, curvature, grades, and lane width. All of these elements are factors in arriving at the completed plan.

The writer feels that Mr. Curtin has confused his discussion, to some extent, by treating both the approach and the approach connection the same. The design of the approach, particularly if it is one of the type of an arterial highway, may be entirely different from that of an approach connection. Physical limitations, such as existing street patterns, buildings, property damage, and other elements, may force the use of a lesser speed, shorter sight distance and steeper grades on approach connections than would be desirable on the main approach. The writer's experience in such matters in congested areas is such as to convince him of the necessity of considering the approach connection as somewhat different from the approach. Again the fact that the entrance of the vehicle from the approach connection on to the approach is generally made after a stop has been required, indicates that the speed limitation should be lower, rather than equal to the speed limitations on the approach. It follows, therefore, that if speed is a minor element in the approach connection, grades may be increased, curvature sharper, and sight distance less. Sharper curves may require wider traffic lanes. As an illustration, in Allegheny County it has been necessary, on approach connections, to have curve radii as low as 80 ft. In such cases the minimum lane width used is 12 ft. Attempt is made to fix the maximum grade at 5%. This gradient is only exceeded when forced by physical conditions.

The writer must take exception to Mr. Curtin when, under the topic of "Approach Connections: Wye Junctions at Connections," he states: "It is desirable to reduce, as much as possible, the slowing down of traffic which results when streams of vehicles divide or join at the intersection of two roadways." It must be remembered that the maximum capacity of any highway is reached at a vehicular speed of 23 to 25 miles per hr. Safety alone should demand a slowing down of vehicles at such wye junctions. Actual practise on most highways properly supervised requires that one of the converging highways be a "Stop" point. The engineer may discuss streams of traffic and fluidity of movement, but the fact will still remain that he cannot mix two streams of automobiles as one mixes streams of water.

The latter part of the paper, covering "Plazas: Provisions for Toll Collection Facilities," "Plazas: Provisions for Convergence of Lanes" and other topics, seems to the writer to cover, fully, the field of the particular topic.

From an engineering standpoint, the paper is excellent. However, to the writer it is only the first of two chapters on the subject. The subject is not completely covered without a discussion of the economics. Substantially all the projects discussed by Mr. Curtin are toll structures, many financed in part by federal grants. For those who must deal in planning projects of similar nature, with or without tolls, without federal "largess," the economics of the problem are as vital as any other element.

Charles M. Noble,[15] M. Am. Soc. C. E. (by letter).—In the past, many unrelated groups of engineers have been called upon to design bridge and tunnel approaches and each group has solved the problems based entirely upon individual judgment without the benefit of being able to refer to comparative criteria. Consequently, the results have reflected this individuality and in many cases undesirable nonuniformity has resulted which could have been eliminated had literature stating fundamental principles been available.

Mr. Curtin is to be congratulated on his able and comprehensive presentation of the factors entering into the proper design of bridge and tunnel approaches. There is little or no literature on the subject and for the first time comparative criteria have been set up in easy reference form for further study by members of the profession.

The paper is notable in that it delineates the fundamental requirements entering into the design of approaches for motor traffic. It treats details sufficiently so that a base line is established from which future designs may be developed, and it furnishes a medium through which this particular art may be developed upon a more scientific engineering basis. In the advancement of any art toward a scientific approach it is first necessary for someone to make available, to the practitioners of that art, literature in which first principles are stated and fundamental criteria formulated. After this is done, and as time passes, these fundamentals may be restated in more precise form and the criteria revised to more nearly approach a perfect state. Only in this manner and through this practical development can principles of design be established on a more scientific foundation, and the writer feels that Mr. Curtin has performed a distinct service to the profession by the presentation of this paper.

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[15] Special Highway Engr., Pennsylvania Turnpike Comm., Harrisburg, Pa.

There can be little doubt that the provision of adequate and ample approaches to a bridge or tunnel is fundamentally sound. The truth of this has been fully demonstrated in almost every large city where vehicular crossings have been constructed without regard to suitable approach facilities and where the economic loss due to incomplete use of the structure itself and to the congestion occasioned in the adjacent street system can be readily seen. To the writer's knowledge, O. H. Ammann, M. Am. Soc. C. E., was a pioneer in recognizing that the design of the approaches to vehicular crossings was of equal importance to the design of the main structure itself and that it was essential for sufficient money to be expended on approaches in order to accomplish, fully, the ultimate objectives of the project. Mr. Ammann enunciated these principles in 1925 during the early stages of the design of the George Washington Bridge, and consistently followed this policy in the design of the subsequent structures with which he was connected as chief engineer. The result may be observed on the George Washington Bridge, the Bayonne Bridge, the Outerbridge Crossing, the Goethals Bridge, the Lincoln Tunnel, the Triborough Bridge, and the Whitestone Bridge—all in the metropolitan area of New York City.

As Mr. Curtin states, the development of a satisfactory approach and plaza system for a bridge and tunnel crossing depends on an exhaustive analysis of all the factors surrounding each particular site, such as traffic volume (direction and intensity), topography, property values, and the presence of utilities, such as railways, subways, aqueducts, and other public services. Indeed, the "cart" must often "come before the horse," for the suitability of contiguous areas for the proper development of an approach system should be determined, in a general way, before the site of the main crossing is selected. In order that the over-all economic health of the community may be preserved most completely, it is most desirable that consideration also be given to the effect of the new facility on adjacent property. Often, complications in the development of approach plans have been the cause of the rejection and elimination of alternate locations from further consideration.

Experience in the past seems to indicate, in general, that elevated structures have a tendency to depreciate surrounding property values; whereas depressed construction apparently has no adverse effect, and in some instances an improvement in property values has occurred after completion of the project. This is understandable when the limitations on light and air imposed by an elevated structure are considered. A correspondingly favorable effect is produced by depressed construction, which involves clearing away buildings and results in an open wide space, thereby improving light and air conditions. This does not mean, however, that elevated construction has no place in the design of highway facilities, but it does mean that the engineer in charge should be fully awake to the broad implications involved, in order that the problem will be studied, not only from an engineering and traffic standpoint, but also from the viewpoint of the economic and social effects on the community at large.

The origin and destination traffic studies, the analysis of the capacity of existing traffic arteries and prognostication of future requirements outlined by the author together with comparative cost estimates, a survey of the probable effect of the improvement on the tax structure, and the social and economic effects on the community in particular and at large constitute the only intelligent method of determining the specific location and the general design of the approach system.

Mr. Curtin analyzes correctly the disadvantages of the type of reservoir plaza in which the toll booths are arranged in semicircular form and, in comparison, stresses the advantages of the tapered plaza. He advances the idea, however, that the tapered plaza evolved from the design of the advance yard in railroad practice. It is possible to apply the reasoning behind the advance yard to tapered vehicular plazas but as a matter of fact the application of the reasoning came after the development. In the case of the George Washington Bridge the general shape of the New Jersey plaza had been determined as early as 1925, which was considerably prior to the application of railroad usage to this particular problem. The tapered plaza is a natural outgrowth of the obvious flaring required by the increased width made necessary by the toll booths. If there is a street or highway crossing adjacent to any form of plaza, sufficient storage space should be provided so that traffic during peak hours does not "back up" across the intersecting street and block it.

It is believed that the relation of width-to-length in designing converging lanes was first consciously recognized when engineers were designing the New Jersey toll plaza for the Lincoln Tunnel. Unfortunately, no comparative criteria were available and it was necessary to proceed by using engineering instinct and judgment, since, to the writer's knowledge, no scientific method had been devised up to that time. Mr. Curtin has performed a service in presenting his analysis. It would appear, however, that the use of $f = 0.15$ for deducing the forward distance required for lateral movement is somewhat high for this type of movement; and, furthermore, the execution of a true reverse curve by a motorist is a practical impossibility. In addition, in a crowded plaza the opportunity must present itself before the motorist can progress in a lateral direction, and often considerable forward distance has been consumed before he has found a "hole" in the traffic of sufficient magnitude to permit the lateral movement. The process is one of constant adjustment, braking, falling in behind a parallel vehicle and moving laterally a lane at a time. All of this consumes more longitudinal space than would be indicated, either by the friction analysis or by a straight passing operation on the open road. It is during periods of heavy concentrated traffic that the value of the length-to-width principle is demonstrated and, if the length is too short for the width, congestion will develop which will effectively block and tie up traffic, thus reducing the economic efficiency of the main structure. When it is necessary to merge traffic into a single lane, more forward distance is required than when multiple lanes are involved, not only because of congestion during peak flows but also to assure reasonable safety. It is also felt that a convergence to a tunnel will require more distance than for a bridge and that the convergence should be completed sufficiently in advance of the portal to assure safety.

On the New Jersey approach to the Lincoln Tunnel, the maximum lateral movement for vehicles converging to enter the tunnel is 35 ft and this is accomplished in a distance of 490 ft, the full convergence being accomplished in the open plaza 100 ft in front of the portal. The average speed from a standing start at the toll booths to the tunnel portal may be assumed at approximately 30 miles per hr. This length may be compared with the length of 243 ft interpolated from Table 2. Unfortunately, only one tube of the Lincoln Tunnel is in operation and traffic has not yet been sufficiently dense to furnish a definite guide in setting up the ratio of width to length. The maximum lateral movement on the toll plaza of the George Washington Bridge is 48 ft and this is accomplished in a forward distance of 510 ft which compares with a distance of 300 ft interpolated from Table 2. The convergence has the appearance of being quite severe and as yet has not been subjected to a full traffic load, since a width of only 57.5 ft (two 28.75 ft roadways each) has been paved out of the 90-ft width available on the main structure. In entering the tunnels of the Pennsylvania Turnpike it will be necessary to merge two 12-ft lanes into a single 11.5-ft lane and, due to the narrowing of the median strip, the lateral movement amounts to 17.25 ft. The minimum forward distance in which this is accomplished is 700 ft, and an additional distance of 100 ft is provided before reaching the portal, making a total distance of 800 ft from the beginning of the convergence to the tunnel portal. It is planned to post the tunnel approaches for speeds of 35 miles per hr. From Table 2, for a lateral movement of 17.25 ft, a forward distance of 274 ft is interpolated, assuming a speed of 50 miles per hr.

It is suggested that additional study be given the subject of convergence in view of its economic importance, and, if required, a new table prepared to supersede Tables 1 and 2.

In connection with the question of grades it may be pertinent to add to the data in the paper that the maximum grade at toll booths should be limited to 2%, with less if possible, because of the tendency of the vehicle to roll while the patron is paying the fare. No grade exceeding 2% was used at the toll booths of any of the structures constructed by The Port of New York Authority.

In making provision for toll collection facilities the usual criterion is based on the peak-hour volume expected during the life of the structure as a toll venture. Many facilities will never attract, during this time, a volume of traffic that will result in an intensity of 1,500 vehicles per lane per hr. The traffic simply may not be available. Again, if traffic forecasts indicate that it will be many years before such a concentration occurs, but that such an intensity will occur during the toll life of the project, provision can be allowed for future expansion of the toll area in the initial design, but at first, only sufficient lanes to accommodate the peak-hour traffic during a period of fifteen or twenty years may be provided. Usually, however, the savings realized from such an expedient are rather meager.

In computing the available traffic capacity of the main structure it may be erroneous to multiply the number of lanes by a value of 1,500 vehicles per lane per hr because practically all authorities agree that, as the number of lanes increases, the lane capacity decreases. In addition, most peak hours are unidirectional and one half of the structure may be congested, whereas on the other half traffic may be comparatively light. It is also true that when high intensities develop, there is always agitation by the public for the construction of additional crossings to relieve the congestion.

The toll area for the George Washington Bridge was computed on the basis of a total volume of traffic during the peak hour of 8,400 vehicles for the nine lanes and on the assumption that each toll lane could handle 400 vehicles per hr. On this basis twenty toll lanes were provided, assuming that the surplus 400 vehicles could be absorbed. As Mr. Curtin has stated, actual operating experience indicates that tolls can be collected at the rate of 450 to 600 vehicles per lane per hr, depending on the toll classification structure. It is possible that in the future traffic may increase to such an extent that the toll area of the George Washington Bridge (twenty toll lanes compared with twenty-seven recommended by Mr. Curtin) will be inadequate; but it is worthy of note that an additional crossing, the Lincoln Tunnel, was constructed before any congestion developed on the structure.

That the width of a toll area may sometimes have an important bearing on the total cost of a project is illustrated in the case of the Lincoln Tunnel where the toll area cost $8,500 per ft of width exclusive of buildings and toll equipment.

In addition to the factors mentioned in the paper relative to the necessity of selecting a design speed, the very fact that a design speed has been adopted immediately permits the engineer to think in terms of seconds of time and to plan the layout on the basis of time rather than in feet. Thus, design elements can be thought of as so many seconds apart, instead of so many feet, and a correlation obtained between the motorist's reaction and the design.

Generalized statements, that superelevation on horizontal curves can be limited to three fourths the design speed and that a friction factor of $f = 0.15$ due to unbalanced centrifugal force is acceptable, should be used with caution. For example, if the superelevation is based on zero friction at three fourths the design speed and a speed of 60 miles per hr is selected, the "balanced" speed amounts to 45 miles per hr and a superelevation of 0.142 ft per ft for a 6° curve will result, which is too great a rate for practical use on public highways. If a maximum superelevation of 0.1 ft per ft is assumed for a 6° curve a friction factor of $f = 0.15$ is developed at 60 miles per hr. On the other hand, when a 2° curve is considered (design speed, 60 miles per hr; and balanced speed, 45 miles per hr) a superelevation of $\frac{5}{8}$ in. per ft is required and a friction factor of 0.03 is developed at 60 miles per hr. In this latter case, the superelevation is so great in comparison with the degree of curve that slower vehicles may be induced unconsciously to "edge in" toward the center of the curve; and the friction developed at 60 miles per hr amounts to only one fifth that developed at the same speed on a 6° curve. Since unbalanced friction is the phenomenon which largely influences the driver of a vehicle it would appear very desirable to design superelevation on the basis of a fairly uniform side friction value in order to offer the motorist uniform and consistent operating conditions on curves.

Tests have shown conclusively that, of their own volition, motorists will develop higher friction factors at low speeds than at high speeds. In turning a corner at 15 or 20 miles per hr they will often develop a friction of 0.25 and 0.30, whereas at higher speeds, when the vehicle begins to "get light on its feet," less and less friction will be utilized as the speed is increased. In a report[16] to the Highway Research Board, Joseph Barnett, M. Am. Soc. C. E., reached the tentative conclusion that friction factors could be utilized in the design of horizontal curves on the basis of $f = 0.16$ up to and including 60 miles per hr with a reduction of 0.01 for each increase in speed of 5 miles per hr for values above 60 miles per hour. This is tabulated as follows:

Speed, in miles per hr	Friction factor "f"	Speed, in miles per hr	Friction factor "f"
60	0.16	85	0.11
65	0.15	90	0.10
70	0.14	95	0.09
75	0.13	100	0.08
80	0.12		

Mr. Barnett added that insufficient tests were available to confirm the trend for speeds greater than 60 miles per hr. Another factor, which the writer feels has an influence on the problem, is the difference in position of the center of gravity of the driver and that of the vehicle. It is believed that this factor will induce drivers to develop higher friction factors on highly superelevated curves than they will on curves with a low rate of superelevation.

In view of the conditions outlined herein it is felt that more tests and confirming evidence are required before authoritative rules can be advanced for curve superelevation.

The writer feels that 10-ft lanes are somewhat narrow on a crowded structure carrying heavy mixed traffic. In the case of two maximum-width trucks each traveling, side by side, in the center of its 10-ft lane, the clearance between the trucks will amount to only 2 ft. Mr. Curtin is quite correct in stating that additional width should be provided in lanes next to curbs. Observations on the West Side Highway in New York City indicate that a large proportion of motorists operate from 4 to 6 ft clear of the right-hand curb. At a considerable increase in cost, lanes 10.75 ft wide were provided in the Lincoln Tunnel, and 11.5-ft lanes have been designed for the seven tunnels on the Pennsylvania Turnpike. In order to obtain the maximum efficiency from a crowded roadway it is essential that the lane lines be properly delineated with pavement markings.

The writer wishes to emphasize all Mr. Curtin has stated with respect to wye junctions at connections and to stress the undesirability of locating junctions close together. Perhaps nothing tends to confuse the motorist more or is the cause of more turbulence than a bewildering maze of junctions close together.

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[16] "Report of Project Committee on Relation of Curvature to Speed," by Joseph Barnett, Highway Research Board, November, 1936.

The desirability of providing tangent alinement at tunnel portals should be emphasized. On the Pennsylvania Turnpike particular attention was paid to this feature and tangents were projected out from the side of the mountain in the open for a distance of 800 to 1,000 ft from the portals in all but three of the fourteen approaches. In the event that curves are required within tunnels it is most desirable for tangent alinement to continue into the tunnel for a distance of not less than 600 ft (preferably 1,000 ft) before starting the curve. This will permit the eye of the driver to become accustomed to the change in light and to be able to see the curve well in advance.

The retinal adaptation of the eye at the entrance to tunnels was given careful consideration during the design of the Lincoln Tunnel and both portals were flared vertically to provide a gradual light change from daylight to the artificial illumination within the tunnel. It is believed that the flare at the New Jersey portal has produced satisfactory results. In New York, however, the shallow clearance beneath Tenth Avenue and the presence of numerous underground utilities prevented a sufficient amount of flare to accomplish completely satisfactory results. The tunnels on the Pennsylvania Turnpike have been provided with a vertical flare equivalent to approximately 29% of the normal ceiling height and with a modicum of auxiliary artificial illumination at the portals. Auxiliary lighting can be wired to separate circuits so that it may be turned off during the hours of darkness, thus relieving the reverse effect of blindness at night when leaving a tunnel. It would appear essential, however, that illumination be provided on tunnel approaches, decreasing in intensity as the distance from the portal increases in order to prevent blindness at night when emerging from a normally illuminated tunnel.

Pedestrian crossings are desirable not only from the standpoint of safety, but also to facilitate the movement of traffic. Irregular groups of pedestrians straggling across a plaza disrupt vehicular movement. When pedestrian travel reaches such proportions that officers must be stationed to halt vehicles for pedestrian passage, the interruption to traffic assumes serious economic proportions when a large volume of vehicular traffic is involved.

In the design of grade-separated pedestrian crossings, the reluctance of the public to using stairways must be recognized and, when possible, inclined ramps with grades not exceeding 10% are desirable. Where steps must be used, fences or barriers should be constructed to prevent pedestrians from crossing the vehicular roadways; otherwise many persons will avoid the steps and continue to walk in the roadway. In considering a choice between an overhead and underground crossing it is well to weigh carefully the advantages and disadvantages of each type before a selection is made. The overhead type will require approximately twice as many steps as an underground passageway because of the required 14-ft vehicular clearance; but it is less likely to be a lurking place for criminals, and will require less supervision. The underground passage requires less steps but, on the other hand, it must be waterproofed, drained, adequately illuminated, and policed during periods when usage is light in order to prevent nuisances and the commission of crime.

In the past design of vehicular crossings there has been a regrettable lack of planning for bus stops adjacent to plazas and, as Mr. Curtin states, this has been the cause of inconvenience, traffic delay, and accident hazard. Each location is a specific problem. Some plazas may never have any appreciable bus-station traffic whereas others, as is illustrated by the George Washington Bridge, will become a concentration depot for bus passengers almost overnight. The author's suggestion of providing, initially, a planned space within which a bus station can be constructed if and when the need arises is the sensible solution to the problem.

In the design of any motor traffic facility, the needs and characteristics of the motor car and the limitations and weaknesses of the driver are fundamental considerations applicable to the general conception and to the design of each individual detail in order that the completed project may be as ideally suited to the use which will be made of it as possible. Mr. Curtin has constantly kept this objective before him in the presentation and has made a valuable contribution to engineering literature.

John F. Curtin,[17] **Jun. Am. Soc. C. E.** (by letter).—The primary premise of this paper is that the approaches to bridge and tunnels govern the capacity and accessibility of these structures. Corollary with this is the premise that the characteristics of the motor vehicle and the inherent limitations of its operator are primary considerations in the design of these facilities.

It is gratifying to note that all of those who have been kind enough to submit discussions on this paper are substantially in agreement with the main contentions. Some of them have contributed additional material to substantiate the premises and to broaden their application. The writer is grateful for these additions as well as for the discussion in which exceptions are made to certain of the design considerations as presented. The latter is more interesting, perhaps, for it reveals the statements wherein the writer has not been entirely clear as well as those upon which a difference of opinion exists. Some further comment on these points is warranted in order to correct the misunderstandings and to clarify the disagreements.

As Mr. Edwards states, the obsoleteness of the approaches to the Brooklyn and Queensboro bridges in New York City, although their main spans are still useful, is evidence of the need for greater consideration upon entrance and exit facilities. The City of New York is now (1940) spending $68,000,000 for the Queens Midtown Tunnel, and plans to spend another $80,000,000 for the Battery Tunnel in order to handle the increased volume of traffic crossing the East River. It is doubtful that either of these tunnels would be necessary if the approaches of the Brooklyn and Queensboro bridges could be developed to their full potential capacities. Both of the new facilities will tap the same traffic sheds that these two bridges, together with the Manhattan and Williamsburgh bridges, are serving. However, the two new tunnels will provide probably a more economical, and certainly a more effective, solution to the East River crossing problem, because the cost of the major operation which would be necessary to modernize the approaches to the older bridges now would be prohibitive.

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[17] Highway Engr., Economist, Am. Petroleum Inst., New York, N. Y.

In a city such as New York, the coordinated movement of traffic is so dependent upon the arterial plan of the city that it frequently is more economical, in serving an area, to build a group of facilities with less extensive approaches than to attempt to converge all traffic upon one or two crossings. As Mr. Herrold suggests, the comprehensive city plan may dictate the necessity of having several facilities spaced some distance apart.

For a large city, the problem resolves into one of determining the number of crossings which, in conformity to the city plan, will afford the maximum convenience in traffic movement with a minimum of backtracking and overlapping, at the same time considering the relationship between the cost of these traffic facilities and the transportation benefits that are derived from them. When this ground work is completed, the approaches of each bridge or tunnel can then be designed so that the crossing will fit into the arterial plan and handle its proportionate share of traffic with maximum convenience.

This arrangement should prevail regardless of how the structure is financed. Mr. Noble states it aptly in his conclusion that the various traffic studies together with the cost estimates and a recognition of the community aspects "constitute the only intelligent method of determining the specific location and the general design of the approach system." For this reason, the writer cannot agree with Mr. Martin's contention that the location of a non-toll project might differ materially from that of a structure to be financed by tolls. Although it is true that topography, land value, and other physical considerations must be weighed in selecting the site of a crossing, fundamentally it is the fitness of the project as a traffic facility, not its cheapness, which determines its value to the motoring public and to the community.

Only too often engineers have been misguided, by shortsighted economy, into selecting a cheaper site or designing an inadequate approach for a structure. This is particularly true on the so-called "free" projects, where the design does not have to stand the "cash register test." Regardless of whether the motorist pays for the crossings directly through tolls or indirectly through gasoline taxes, it remains the responsibility of the engineer to determine the location of the structure and the design of its approach system that will accommodate the coordinated movement of traffic in an economic manner—without compromising that objective.

The reason that the writer has dealt mainly with toll structures is, as Mr. Herrold states, that "more attention has been given to approaches where tolls are charged than to approaches to tunnels and bridges which have been paid for through taxes." Although the economic considerations have been treated only incidentally in this paper, due to limitations of space and more adequate treatment by other writers, it is evident from inspection of them that the approaches of toll structures are superior economically as well as functionally to those of non-toll projects. The self-liquidating facilities have a better ratio of total motoring benefits to total costs since they are scrutinized more carefully in the planning stage from that angle in order to insure solvency. The implication by one writer that federal "largess" has obviated economic considerations with respect to toll projects is totally unfounded.

However, there is no real reason why the same principles of economic analysis cannot be applied to non-toll projects. Heretofore, engineers have relied mainly on a comparison of cost estimates in determining which in a group of approach studies for a bridge or tunnel is most economical. The various items of expense, property, construction, maintenance, and operation are estimated for each plan, and since the total of these is the most tangible sum that can be assigned to that proposal, it becomes the basis of comparison. Nevertheless, the inadequacy of comparing only the total costs is well recognized, for the final selection is generally made by applying a judgment factor, which embraces an understanding of the other considerations, to the comparative costs. Furthermore, although most bridges and tunnels indicate that the final selection of site and approach development was the most satisfactory solution, there are many which show that one or more of the intangible considerations were slighted or neglected.

If the full return is to be had upon the investment in a bridge or tunnel, a more inclusive analysis must be used in determining the proper location and development of the approaches. As Mr. Martin states, "the final decision * * * must be based on the functional adequacy of the device, with due consideration to the cost." Although it is more difficult to prepare than the cost estimate, an analysis of the potential traffic and the benefits that will accrue to it by use of the proposed facility should also be made in terms of monetary value. Nathan Cherniack, Assoc. M. Am. Soc. C. E., has developed an effective method for measuring the potential traffic of a proposed vehicular crossing in terms of transportation benefits which he described in an excellent paper.[18] In addition, John Beakey and C. B. McCullough, M. Am. Soc. C. E., have derived a set of monetary relationships among the cost of a highway facility, its potential traffic, and the benefits which it will provide for that traffic.[19] By considering all items on the common denominator of dollars and cents, the correlation between factors may be seen prosaically and a concrete index of the value of each proposal may be had. That site and its approach development which has the highest ratio of benefits to costs is the most desirable from the standpoint of the motoring public, whether a toll project or not. For a toll structure, the plan offering the greatest over-all benefits will attract the most income to the facility; for a non-toll structure, it will serve the public best. Although tolls are an important financial consideration, they are only a means of financing and, therefore, bear no direct relationship to the physical planning other than to require certain provisions for collecting them.

The conclusion drawn by Mr. Beretta that only the approaches of large bridges and tunnels have received proper attention in design is generally true, although there are some excellent examples of approach development on smaller structures. As a rule, the traffic design problems on a small vehicular crossing are not as complex as those on a large bridge or tunnel and, therefore, the approaches should be more adaptable to the ideal solution. However, the relative size of these crossings and the fact that frequently the design of such structures is standardized in many respects tend to lessen their importance as traffic facilities so that their approaches are more likely to be neglected. Messrs. Beretta and Herrold concur in the opinion that small river crossings could be improved materially and extended in service life, if more consideration were given to these structures from the standpoint of traffic design.

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[18] "Measuring the Potential Traffic of a Proposed Vehicular Crossing," by N. Cherniack, *Proceedings*, Am. Soc. C. E., February, 1940, p. 207.

[19] "Economics of Highway Planning," by John Beakey and C. B. McCullough, *Technical Bulletin No. 7*, Oregon State Highway Commission, 1937.

Apparently, some difference of opinion exists as to the usefulness of circles for regulating traffic on bridge and tunnel approaches in cities. Mr. Martin states that they have been economical and satisfactory in his experience, whereas Mr. Edwards demonstrates their ineffectiveness in handling traffic in Washington, D. C. The writer is inclined to agree more with the conclusions of the latter—that traffic circles should be limited to rural and suburban regions.

Because of their successful adaptation to the intersections of heavily traveled highways, the installation of circles as approaches became popular in the late 1920's. In quick succession, the planning engineers of the Goethals Bridge, Outerbridge Crossing, and the Delaware River Bridge adopted the rotary principle for one approach of each of these structures. Its advantage over the street extension and reservoir types of approaches was the presumption of continuously moving traffic and the elimination of the bothersome left turn. By compelling all vehicles to proceed in one direction around a circle, the approach became capable of handling a larger volume of traffic with less delay. The theory of operation is that conflict is minimized by the substitution of a continuous weaving operation by vehicles moving in the same circular direction for a direct crossing. This principle is discussed fully by Guy Kelcey, M. Am. Soc. C. E., in his excellent paper.[20]

The size of the central island has a considerable influence on the efficiency of a traffic circle because the radius of the roadway must be large enough to accommodate a reasonable speed, and sufficient distance must be provided between connections to the circle for the interweaving of vehicles. This latter factor tends to slacken the speed and reduce the capacity because of the gaps created by cars weaving in and out on the circular roadway. The necessity of moving across the circle causes a greater heading between vehicles and consequently reduces the lane capacity. In addition, traffic entering and leaving the crossing frequently must merge on one or two quadrants of the circular roadway. This induces additional interference and further reduces the effective capacity of each lane. Furthermore, the circling operation often causes motorists to become confused and to turn off at the wrong roadway.

Rotaries are being used with moderate success in undeveloped areas; in cities, however, the waste of land in the central island makes them impractical. This island cannot be occupied without seriously impairing the operation of traffic and may be used only for decorative statues or landscaping. Pedestrian access creates a dangerous accident hazard and vehicular access invites operational interference as well as the loss of one lane due to parking. Furthermore, as Mr. Edwards states, circles operate ineffectively under heavy traffic and do not obviate the use of lights.

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[20] "Channelization of Motor Traffic," by Guy Kelcey, *Proceedings*, Am. Soc. C. E., December, 1939, p. 1649.

Virtually the same criticisms may be made of the reservoir approach, for on this type, the unwieldy maneuvering between the rectangular plaza and the roadway of the bridge or tunnel reduces the effectiveness of the facility unless a tapered area is provided between them, in which case, the rectangular space is largely unnecessary. Furthermore, the rectangular area is not fully used, because it is ill-adapted to vehicular movement; as much as 40% of it is wasted as far as storage space is concerned. This is illustrated at the Holland Tunnel Approach, Fig. 16, to which the paper and Mr. Martin's comment specifically referred. Unlike a parking lot, the storage space of an approach is used only momentarily for that purpose, and therefore it should be shaped so as to accommodate the continuously converging operation of vehicles.

Notwithstanding the comment by Mr. Martin that circles are especially adaptable to flat areas, it has been the observation of the writer that the most effective traffic circles are those in which the third dimension, height, has been utilized. One example of this is the circle at the Rockaway Beach end of the Cross Bay Bridge in Queens, New York. The rotary is elevated above the roadway which intersects the bridge approach and, as a result, is used only by traffic going on or off the bridge. Traffic on the intersecting road continues in a straight line without having to mingle with bridge traffic, and adequate turnouts are provided so that vehicles may turn off to the bridge or from the bridge to the road, without interrupting the main stream. Even when the span of the bridge is raised, there is sufficient space to store the bridge vehicles on the elevated circle so that traffic on the intersecting roadway can continue unhindered.

Another traffic circle that is unusual in that height was used for its efficacy is the three-level structure on the New Jersey Approach of the Lincoln Tunnel (Fig. 17). This development of the rotary principle goes beyond that of the aforementioned Cross Bay Bridge, for in this case the traffic on both the depressed approach and the intersecting avenue continues on tangent alinement without interruption. The circle, which is the intermediate level, is used only by vehicles turning from one roadway to another. The disadvantages of rotary traffic are overcome entirely on this structure because no weaving is required on the main roads, except possibly at the turnouts and, therefore, traffic is not slowed down. The radius of the circle is small so that comparatively little land is required, and yet it is large enough to accommodate the interchanging vehicles at the speed with which that operation is made. This type of grade separation is very well adapted to the intersection of express highways where the cost of land is high and where plenty of clearance between the two roads is available. It requires much less property than a clover-leaf structure and is much less confusing to drive upon, but a difference in elevation of approximately 40 ft between the main roadways is necessary.

The writer is grateful to Mr. Beretta for his valuable contribution regarding the approaches of international structures. The special problems which port-of-entry inspection presents are complicated not only by the various types of traffic, each requiring a different type of inspection, but also by the multiplicity of agencies making these inspections. As Mr. Beretta suggests, how-

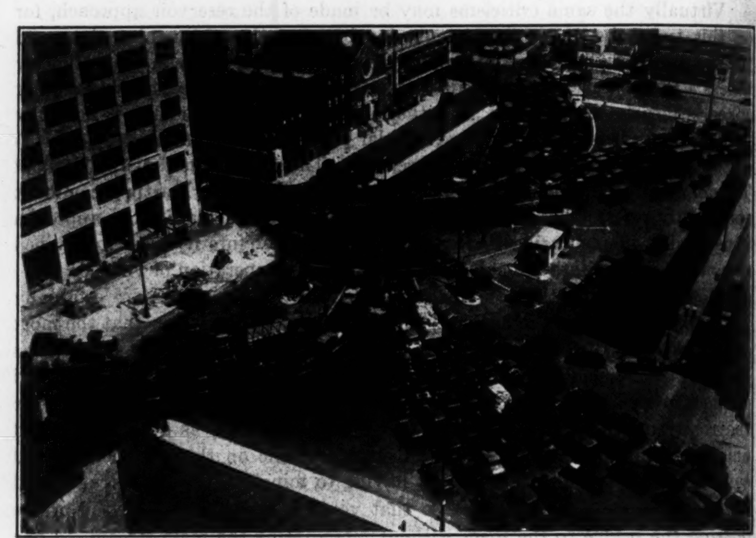

Fig. 16.—Peak Traffic at New York Approach, Holland Tunnel, New York, N. Y.

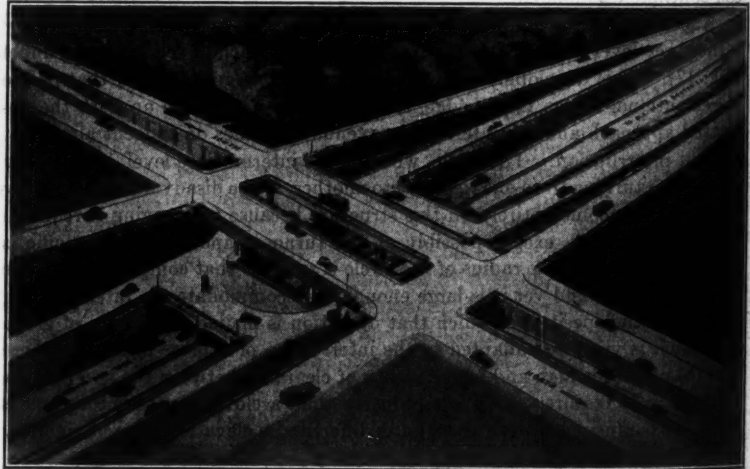

Fig. 17.—Three-Level Traffic Interchange at New Jersey Approach, Lincoln Tunnel, Union City, N. J.

ever, these problems are capable of solution by the application of known principles of traffic design, together with those adopted from rail and water transportation.

Origin-destination studies should be especially helpful in designing international bridge approaches for they provide the key to the classes of traffic from which the degree of inspection required for each type may be determined. They also indicate the peak loads so that the capacity requirements may be estimated. With this information, the various traffic movements can be segregated among the inspection stations in a manner similar to the way in which they are distributed geographically from any other bridge approach. The remaining problem then is one of arranging and, if possible, consolidating the inspection facilities so as to require a minimum of conflict among the various traffic movements.

Mr. Hartley has made a valuable addition to the paper by his discussion on toll-lane capacities for small structures. Most of the toll stations at the various interchanges on the Pennsylvania Turnpike were arranged in the manner described by Mr. Hartley in Fig. 9(*c*).

In analyzing the capacity requirements for a small toll station, the low traffic-volume periods should be considered as well as the peaks in order that an excessive number of collectors will not be required. The layout of the Market Street Bridge in Harrisburg, Pa., is subject to criticism in this respect for it requires two men at all times to collect tolls, whereas one man could handle the traffic during the early morning hours if the booth arrangement permitted it. It is the writer's opinion that a double-faced booth should always be placed in the center of the toll plaza so that in low-volume periods the outer lanes can be closed and all traffic handled by one collector. Since the total traffic volume is likely to be less than 600 vehicles per hr even on a large structure for as much as 25% of the time, it is well to consider this period as much as the peak.

The toll-booth arrangement presented by Mr. Hartley in Fig. 13 allows considerable freedom of traffic movement but it is also subject to the same criticism of inflexibility in operation. A similar set of conditions prevailed on the layout of the Carlisle Interchange of the Pennsylvania Turnpike, but instead of being staggered, the booths were placed straight across the pavement, as shown in Fig. 18. Although this arrangement has only about 60% of the capacity of that shown in Fig. 13, it has the same advantages of minimum lateral movement, no extra graded width and, in addition, it may be handled by one man during low-volume periods. Furthermore, the separate booth for each lane not only permits faster collections in each lane but it also minimizes the inconvenience to the patrons since the tolls are always collected from the left-hand side of the vehicle.

The writer believes that Mr. Martin has perverted his splendid analogy with respect to wye connections that one "cannot mix two streams of automobiles as one mixes streams of water." A wye connection may be considered the same as a wye fitting in a pipe line; the smoother the connection is made, the less turbulence (traffic congestion) will be caused by the fitting. If vehicles must slow down or come to a halt at junctions, as Mr. Martin claims, why were the accelerating and decelerating lanes provided on the approaches of the Highland Park Bridge (Fig. 14) which he cites as a good example of approach design? This same layout was used at the interchanges on the Pennsylvania Turnpike, and the accelerating and decelerating lanes were designed so that vehicles could enter and leave the main roadway at 60 miles per hr. Mr. Noble, who designed these interchanges, emphasizes the point that wye connections should not be located close together because of the turbulence created; and, as Mr. Kelcey shows in his paper,[20] it is not the absolute speed of vehicles but the relative difference in speed that causes accidents. Actually, there is a greater accident hazard in entering a main roadway from a standing start than in merging at the same speed as that of the main stream. The reason that vehicles are required to stop at intersections and some wye connections is not because stopping itself is safer, but because adequate sight distance has not been provided at the connection which makes it necessary for entering vehicles to stop.

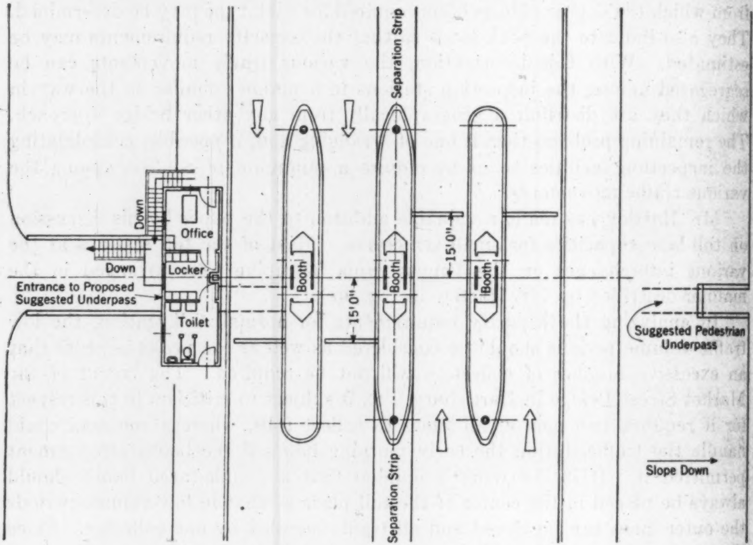

FIG. 18.—TOLL BOOTH LAYOUT AT THE CARLISLE INTERCHANGE, PENNSYLVANIA TURNPIKE, CARLISLE, PA.

The writer agrees entirely with Mr. Martin on the need for wide separating islands on approach connections with the steady flow system. As found by the American Association of State Highway Officials,[21] the minimum width of separation that will enable a passenger car to make a U-turn with reasonable safety is 39 ft, whereas the minimum width for trucks is 71 ft. In general, it is desirable to make the separating islands as wide as possible. However, the cost of right of way, particularly in urban or suburban areas, frequently makes it necessary to design these islands with less than the desirable minimum width.

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[21] "A Policy on Highway Types," by Am. Assoc. of State Highway Officials, 1940.

Most sincere thanks are due to Mr. Noble not only for the valuable additions which he has brought to consideration but also for his efforts in applying some of the conclusions of the paper to practical test.

The writer agrees wholeheartedly with Mr. Noble on the importance of considering the influence of a bridge or tunnel approach upon adjacent property and the community. Only too often the broad, social and economic implications have been neglected, because of the desire to secure a low-cost solution to a bridge or tunnel approach problem. The effect of a bridge structure upon property value and the limitations of light and air imposed by such a structure were brought out fully in the controversy over the proposed Battery Bridge in New York City.

Mr. Noble's diagnosis of the factors involved in determining the ratio of length to width of plazas is very illuminating, although the writer does not agree that the use of $f = 0.15$ is high in determining this ratio. Mr. Noble states further that a friction of 0.25 to 0.30 frequently is developed at 15 or 20 miles per hr, but that at higher speeds less and less friction is utilized. Although it depends entirely on the design speed, the value given in the paper should be ample for most plazas, particularly since the table cited by Mr. Noble gives a permissible design speed of 65 miles per hr for $f = 0.15$.

Around this same point hinges the difference in the writer's and Mr. Noble's use of the tables for determining the forward distance required for lateral movement on plazas. The paper states that the design speed should be used in interpreting these tables, whereas Mr. Noble uses the posted operating speed, which is considerably lower, in applying the tabular values to the plazas of the George Washington Bridge and Lincoln Tunnel. The design speed on the plaza should be the same as that for the crossing proper for the reason given in the paper that "the speed of off-bound vehicles approaching the toll booths is the governing criterion." Mr. Noble is correct in his statement that on tunnel plazas the convergence should be completed 100 ft in front of the portal but it should be noted also that the length for convergence is measured from the ends of the toll islands rather than from their center line. Therefore, applying the tables to the New Jersey plaza of the Lincoln Tunnel, a lateral movement of 35 ft at a design speed of 50 miles per hr gives a minimum length of approximately 400 ft which compares favorably with the 450 ft that is provided on this plaza between the ends of the islands and the point of convergence. Similarly, on the George Washington Bridge toll plaza, the application of these tables at a design speed of 50 miles per hr gives a convergence length of 460 to 500 ft for a 48-ft lateral movement, whereas the actual length provided between the ends of the toll islands and the point of convergence is 490 ft. Considering the limitations of the tables and the instinctive design of these two plazas, both examples check the tables remarkably well.

Despite these comparisons, however, Mr. Noble is quite correct in his statement that the subject of convergence deserves additional study because of its economic importance. This writer cites the cost of the Lincoln Tunnel toll plaza at $8,500 per ft of width and it is a certainty that a comparably high value would be developed if the cost per linear foot of length were derived.

Although these tremendous costs tend to restrict the length and width of plaza to a minimum, there should be some assurance that sufficient space is being provided for vehicles to perform the required movements.

The limitations in using convergence tables such as those presented in the paper should also be recognized. These values cannot be applied to the approaches of the Pennsylvania Turnpike tunnels because these facilities are not plazas essentially. When Mr. Noble designed the Turnpike tunnel approaches he made an entirely different analysis which was more befitting the conditions under which vehicles are operating on them. The operating conditions on these approaches are similar to those at a main line switch on a railroad, whereas the plaza might be compared with a classification yard.

It is believed that Mr. Martin has misunderstood the writer's meaning of an approach in the statement that "Mr. Curtin has confused his discussion, to some extent, by treating both the approach and the approach connection the same." Without denying any confusion, it is the writer's feeling that they are the same, or at least that the connections are a component part of the approach. If a bridge or tunnel is to handle its full share of traffic with maximum convenience it should be designed as a whole, not as a patchwork of dissociated parts. Otherwise it never will fit together properly. The design speed and the various other design specifications should control the layout of the entire approach, including the connections. Although it may be necessary to use a different design speed on various parts of an approach to suit the operating or the physical requirements, it is most desirable to have the design unified and as uniform as possible throughout. Any changes should be made only with adequate transitions so that traffic may flow smoothly without suddenly being confronted with different operating conditions, for it is generally recognized that this is one of the primary causes of accidents.

Some disagreement prevails as to maximum lane capacity and the conditions under which it is developed. Mr. Noble takes exception to the value of 1,500 vehicles per lane per hr as being high in some cases, whereas Mr. Hartley cites an instance in which 1,800 vehicles per hr were counted in a single lane and also quotes values in excess of 1,800 per hr at speeds of 35 and 40 miles per hr from the Johanneson capacity formula. In addition, Mr. Martin states that the maximum capacity is reached at a speed of 23 to 25 miles per hr.

Unfortunately, there have been no studies which indicate conclusively the maximum capacity of a traffic lane, although enough work has been done to show definitely that many of the current formulas and notions concerning capacity are erroneous. From arbitrary formulas used in the past, the thought has prevailed that the maximum capacity of a lane is obtained when vehicles are moving uniformly at a speed of 23 miles per hr. Field studies conducted by the U. S. Public Roads Administration[22] have disproved this theory, however. They indicate that the theoretical maximum number of vehicles which one lane will accommodate in an hour is approximately 2,400 when the vehicles uniformly spaced are traveling at 31 miles per hr. Furthermore, these studies show that the theoretical maximum does not fall more than 20% below this value for speeds between 20 and 50 miles per hr.

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[22] "Preliminary Results of Highways Capacity Studies," by O. K. Normann, *Public Roads*, Vol. 19, No. 12, February, 1939.

It is definitely pointed out, however, in the conclusions of these field observations that such capacities are obtainable only in theory and that working capacities are well below these limits. It was more or less in recognition of these conclusions that the writer arbitrarily used a maximum of 1,500 vehicles per hr because, although there is no theoretical basis for it, this value is within the practical limits that have been observed. As a practical matter, neither maximum nor working capacities can be determined to such a nicety from consideration of vehicular speed alone, for the interference and differential speed between vehicles as well as the number of lanes and other design factors have a material influence upon capacity.

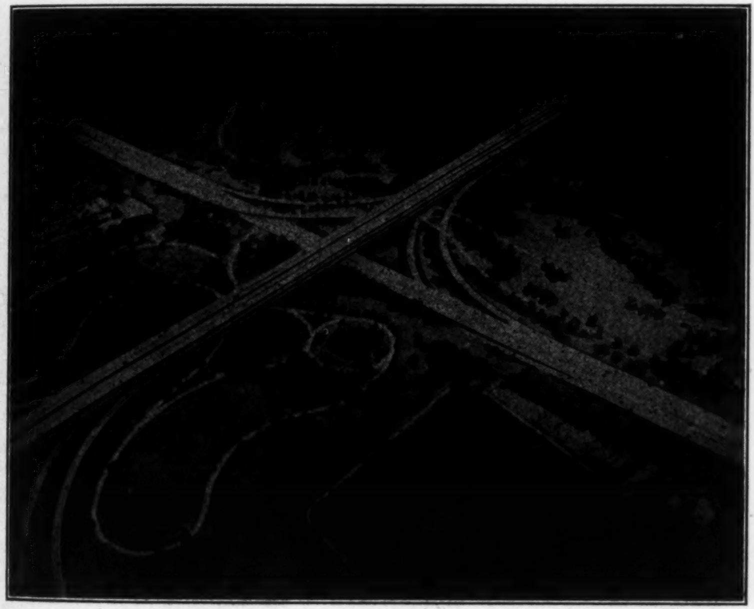

FIG. 19.—BRIDGE APPROACH PREDICTED BY ARTIST FOR TWENTY YEARS HENCE

Although it is not recommended necessarily, it is stimulating, nevertheless, to see the visionary conception of a bridge approach (Fig. 19) which Norman Bel Geddes predicts for 20 years hence.[23] The approach comes down gradually over an extensive area with long, flat curves branching from each of four levels. The principle is sound, but it is hoped that the profession will develop a more economical and less pretentious approach than that which this artist portrays.

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[23] "Magic Motorways," by Norman Bel Geddes, Random House, Inc., New York, 1940.

Considering the progress made over the 20 years 1920 to 1940, it very likely will, for the fundamental principles have been established, examples are available to demonstrate them, and most of the factors involved have been evaluated. The experience of these twenty years is fruitful and all is ready now to be refined and molded to the desired objective. To this end, these meager efforts are dedicated—that some day the motorist in coming home at the weary end of a holiday will not be subjected to "that most dismal of all motoring experiences: driving back to the city across a bridge."[23]

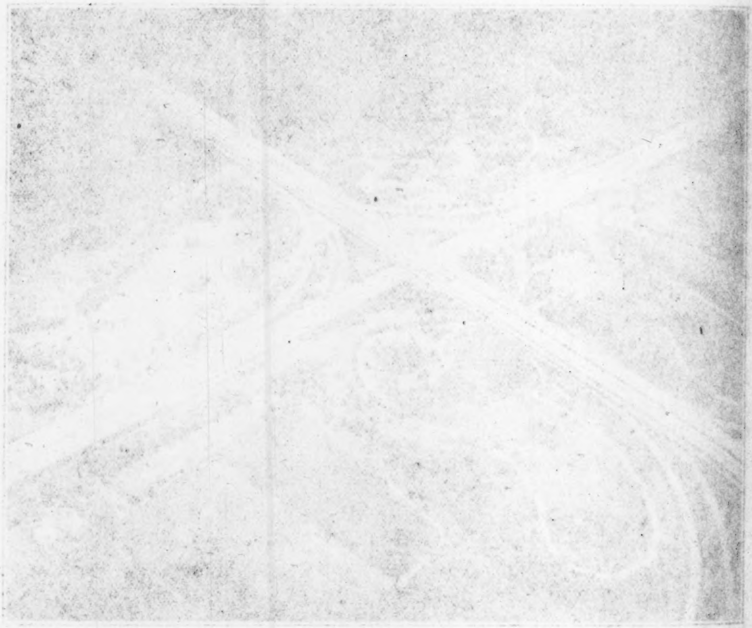

Fig. 101.—[illegible] Approach [illegible] for Traffic Twenty Years Hence.

Although it is not recommended necessarily, it is stimulating nevertheless to see this visionary conception of a bridge approach (Fig. 101) which Norman Bel Geddes predicts for 20 years hence. The approach comes down gradually over an extensive area with long, flat curves branching from each of four levels. The principle is sound, but it is hoped that the profession will develop a more economical and less pretentious approach than that which this artist portrays.

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[23] "Magic Motorways," by Norman Bel Geddes, Random House, Inc., New York, 1940.

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Paper No. 2105

TREND IN HYDRAULIC TURBINE PRACTICE

A SYMPOSIUM

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Note.—Published in November, 1939, *Proceedings*.

ECONOMIC PRINCIPLES IN DESIGN

BY I. A. WINTER,[1] ASSOC. M. AM. SOC. C. E.

SYNOPSIS

New and interesting advancements in hydraulic turbine practice and research, many features of which are still in the proposed or experimental stage, are presented in this paper. The subjects cover a wide range and are treated only in their broadest aspects, based on the experiences of the writer and his associates; it is not intended to present an exhaustive treatment of any subject or to cover the experiences of other qualified experts in the field.

It is hoped that the subjects presented will invite others to contribute constructive data or experiences which will add to the interest of the paper and point toward the future trend in hydraulic turbine practice.

INTRODUCTION

Advancements in the application of hydraulics to the design of power plants have not been all that was desired because there has been too little appreciation of the gain in efficiency, and the saving in first cost, made possible by the application of fundamental elementary principles. In general, the designer has attempted to simplify the field construction by approximating shapes that conform to the ideal shapes. He has reasoned that refinement in design was not necessary; and, as a result, the value of the theorist has been minimized as being impractical.

Progress is now being made toward improved designs and reduced cost, stimulated in some degree by the research in aerodynamics which is being conducted by the airplane industry. In that field the speed and load capacities have been increased steadily by lending attention to fine details. In the future, progress in water-power designing will be parallel to the progress in aeronautical design, inasmuch as specific speeds and velocities now in common use will be increased, resulting in higher efficiency, lower maintenance, and reduced first cost.

TURBINES

In turbine design the trend is toward larger sizes, of which the Wheeler Dam in Alabama, Bonneville Dam in Oregon, and Boulder Dam in Arizona and Nevada, are outstanding examples. At Wheeler the turbines (with fixed-blade propellers) deliver 45,000 hp at 85.7 rpm, under a head of 48 ft. The turbine runner is 22 ft in diameter and discharges 10,600 cu ft per sec at full gate. The Bonneville turbines are fully automatic Kaplan wheels rated at 66,000 hp under 50-ft head, with a five-bladed runner 23 ft 4 in. in diameter, operating at 75 rpm. At the rated heads and power each wheel discharges

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[1] Senior Engr., U. S. Bureau of Reclamation, Denver, Colo.

13,000 cu ft per sec. Complete tests have been made on the wheels, and the results demonstrate that the intake, scroll case, turbine, and draft tube are well designed. Efficiency appears to be (and capacity is) above expectation, with freedom from hydraulic disturbances. The turbines at Boulder Dam are rated at 115,000 hp, under a head of 475 ft, and they operate at 180 rpm. The maximum discharge is approximately 2,500 cu ft per sec when delivering full power at full gate. Under test conditions these units have delivered 100,000 kw for a short interval at a head of 520 ft, which is approximately 130,000 hp in a single-stage runner.

Many improved features of turbine design are contributing to better performance of the units and to reduced maintenance cost. Venting the center of the draft tubes of fixed-blade Francis type, and propeller runner turbines, with free air through the hollow bore in the turbine shaft, similar to that at the power plants at Boulder Dam and Drop No. 4 of the All-American Canal system in California, is effective in eliminating hydraulic shock in the draft tube caused by vortex cavitation. The admission of free or compressed air into the draft tube is effective in obtaining smooth operation of the unit, regardless of the tailwater level. The quantity of air required to quiet the draft tube in this manner is relatively small and has been found to have no adverse effect on the efficiency or horsepower of the turbine at any gate opening.

Water-lubricated bearings of rubber, lignum-vitae, or non-metallic composition are meeting with favor for propeller-type turbines on both the fixed-blade and the movable-blade types. Provisions for the adjustment of the runner clearances are customary. By using water-lubricated bearings in lieu of oil-lubricated babbitted-type bearings, the inaccessible stuffing box below the bearing is eliminated, and the turbine pit may be drained entirely by gravity. Corrosion-resisting steel sleeves are fitted to the shaft opposite the bearing and stuffing box.

Improvements in the type of gate linkage arrangement have been of importance. A design that incorporates a cast-iron brake element in tension, regardless of the direction of operation, as well as gate-limit stops, independent of all associated parts, has been furnished for the Drop No. 4 plant and is of a type similar to that used extensively in Europe.

Progress is being made in the elimination of pitting on runners by reducing the overhang of gates over the curved entrance to the throat ring, finishing the entrance and discharge edge of the blades, and introducing improved blade shapes of simple curvature. On several new projects, consideration is being given to welding a ⅛-in. layer of stainless steel on the runner shroud and low-pressure areas of the vanes of a Francis runner in the manufacturer's shop. Under these conditions the steel coating can be applied at less cost than when applied in the field, and the reduced friction losses, obtained by the use of stainless steel with ground surfaces, will increase the power and efficiency of the unit.

Increased attention is being given to the prevention of vibration, noise, and instability of operation. At least eight distinctly different types of trouble from this source have been encountered and satisfactory solutions found. This is a broad subject, involving many phases of power-plant designs, and offers a fruitful field for further investigation.

One of the outstanding developments in turbines has been the automatic device by which the turbine blades can be moved without hydrostatic pressure from the governor oil system. The blades are designed so that they are pivoted ahead of centers of pressure and are proportioned so that they have an inherent tendency to adjust themselves to the water flow. The adjusting mechanism is concentrated in the runner and turbine pits and is not connected with the governor pressure system. The blades open when the turbine is started to reduce hydraulic thrust, and run-away speed is reduced if the machine should run away at normal head. This unit is a distinct American development.

The head limit for the propeller type of turbine is being increased. Low-speed propeller runners, with from eight to ten blades, have been developed for high heads. This improvement extends into the field of the Francis turbine for specific speeds from 80 to 100, and thus it bridges the gap between the medium-speed Francis turbine and the high-speed propeller turbine. The characteristics of this type of turbine are better and the efficiency is greater than either the high-speed Francis or the usual propeller type.

Laboratory tests on movable-blade propeller turbines have demonstrated that increased power and efficiency at the larger gate openings may be obtained without the necessity of making the bottom of the throat ring spherical. It has been general practice, in the past, to maintain close clearance throughout the periphery of the runner blades when they are opened. The spherical throat reduces the capacity of the runner and causes undesirable eddy formation at the periphery of the draft tube immediately below the runner. It is also conceivable that vortices set up by the wicket gates will have a tendency to collapse on the bulged part of the spherical throat ring, causing increased pitting.

CENTRIFUGAL PUMPS AS TURBINES

Laboratory investigations conducted in 1939 by the California Institute of Technology at Pasadena, Calif., under the direction of the United States Bureau of Reclamation in connection with the pumps for the Grand Coulee pumping plant at Coulee Dam, Washington, emphasize the fact that, when operating in reverse as a turbine, a centrifugal pump may be slightly superior in efficiency to a hydraulic turbine designed as such. Such a finding may be quite consistent with the design principles as the friction surfaces in a centrifugal pump are reduced to a minimum. The blades are smaller in number, the direction and changes of flow are less abrupt, and the leakage area of the seal is reduced by placing the seal rings at a smaller diameter than is possible in the conventional turbine design. The significant fact revealed by these tests is that when the pump is operating in reverse the speed and power generation are comparable to that of the pump, thus making an ideal unit for pumped storage projects. The efficiency of the unit when operating as a turbine or pump under the net effective head is approximately the same.

Many years of research and study have been devoted to designing a turbine that could be used as a pump; but the belief that pumps were inherently low in efficiency precluded any serious research to develop the pump as a turbine. The tests of the pumps for the Grand Coulee pumping plant indicate a definite increase of pump efficiency with an increase of specific speed to a value of approximately 2,500, which is in substantial agreement with the relation of capacity and specific speed to efficiency as shown[2] in Figs. 1 and 2. Fig. 1 shows the optimum efficiency as a function of the capacity of the pumps. This characteristic is not apparent with respect to an increase in specific speed as shown in Fig. 2 where the specific speed N_s is represented by

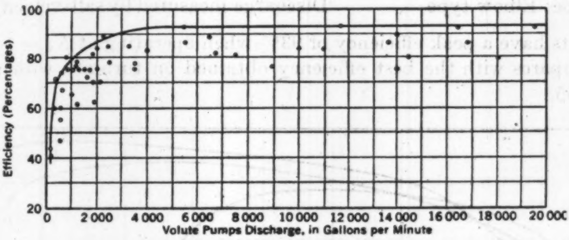

Fig. 1.—Optimum Efficiency as a Function of Capacity

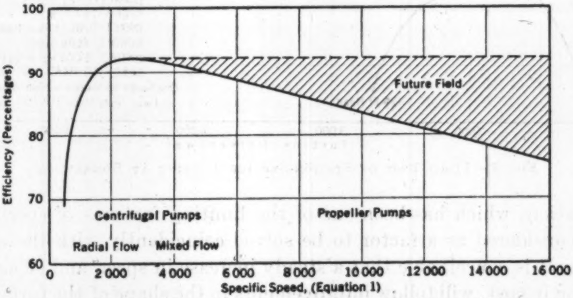

Fig. 2.—Optimum Efficiency as a Function of Specific Speed

$$N_s = \frac{N\sqrt{Q}}{h^{3/4}} \qquad (1)$$

in which N = speed; Q = flow, in gallons per minute; and h = hydraulic head. The shaded area marked "future field" is predicated on the assumption that the increase in demand for pumps of higher specific speeds will result in increased performance similar to recent advancement in the characteristics and efficiency of the propeller-type turbine.

In general, the relation of efficiency to specific speed for turbines is approximately parallel to that for centrifugal pumps when expressed in the same unit, maximum efficiency occurring at $N_s = 2{,}500$ for single-stage, single-suction pumps; and, $N_s = 55$ for similar hydraulic turbines. This relation is being modified as will be noted by reference to Fig. 3, showing the performance of the station-service turbines tested at the Bonneville project from February 5 to 14, 1938. The turbines may be described as follows:

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[2] "Hydraulics," by R. L. Daugherty, McGraw-Hill Book Co., Inc., New York, N. Y., and London, England. 4th Edition, 1937, Figs. 315 and 316.

Type: Kaplan, five blades
Diameter: 81 in.
Speed: 257 rpm
Draft Tube: Elbow type
Scroll: Plate steel
Rating: 5,000 hp at 50 ft head
Specified Head: 30 ft to 80 ft
Discharge measured by salt-velocity method

These units have a peak efficiency of 93% when operating at $N_s = 140$. This value compares with the best efficiency obtained on turbines with a specific speed of 55.

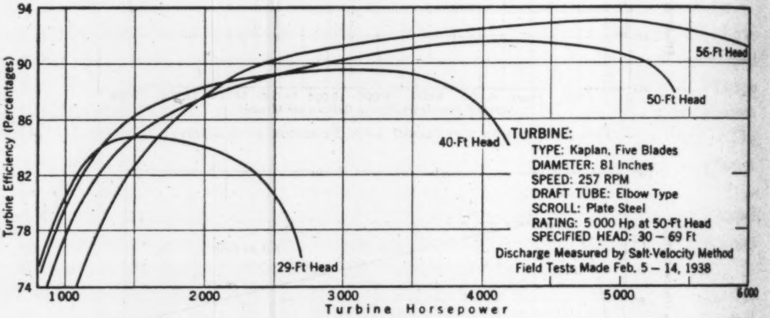


FIG. 3.—FIELD TEST OF STATION-SERVICE TURBINE AT BONNEVILLE

Cavitation, which has been one of the limiting features of specific speed, must be considered as a factor to be solved coincidently with the associated problem. It is conceivable that a steady increase in speed and efficiency, and a reduction in cost, will follow improvements in the shape of the turbine runner vanes and venting of the draft tube, and by the use of hard metals with a high yield point to resist cavitation.

The pumped storage possibilities are exceedingly great for many of the present developed properties where the reservoirs and transmission lines are now in service. The generation of power on the large canal systems which are now being built (1939) throughout Western United States can be made of great economic value by the location of daily pumped storage plants at intervals along the canal system. The All-American Canal system in the southwestern part of the United States will require a seasonal flow variation from a minimum of approximately 3,000 cu ft per sec to a maximum of approximately 6,000 cu ft per sec. A typical power study for this project is shown in Fig. 4. A canal system of this nature does not provide even hourly storage to meet the varying daily load curves; hence, a turbine selected for this class of service must be capable of delivering only base-load power which, in many cases, results in a loss of 20% in the generation of secondary power for which the market has no particular use.

By applying the pumped storage principle to constant varying flows in extended canal-system projects, an ideal power set-up can be created, whereby the economics of the project may be tremendously increased and the hydraulic

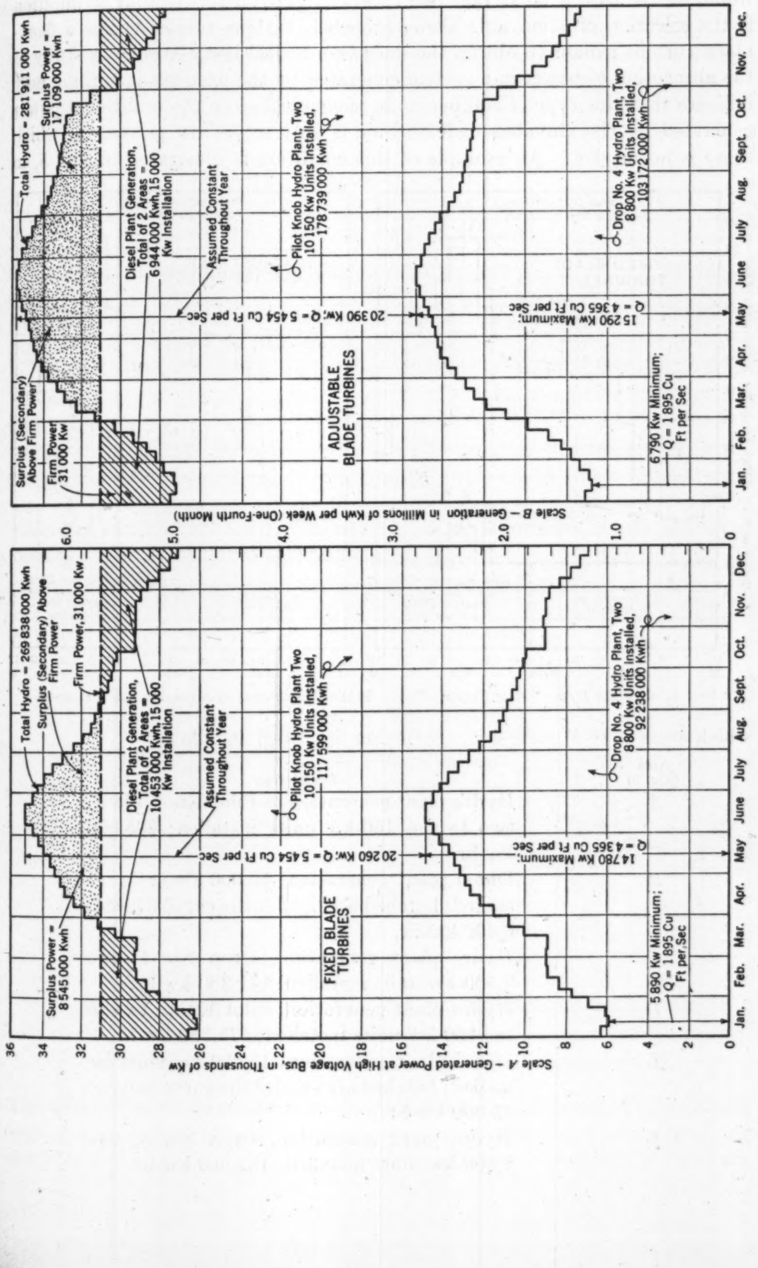


FIG. 4.—SYSTEM YEARLY POWERGRAPH—USING FIXED-BLADE AND MOVABLE-BLADE TURBINES

installation in the power plant may require an entirely different basis than would be the case if no storage facilities were provided. A point in question is the selection of a movable-blade, propeller, turbine runner versus a fixed-blade turbine runner to obtain the most economical installation. A study of the absorption of power in a system generated by the fixed-blade turbine may indicate that this type of equipment is economical since the additional power generated by the movable-blade turbine is only secondary power, for which there is no market. An example of this condition is illustrated in Fig. 5, in

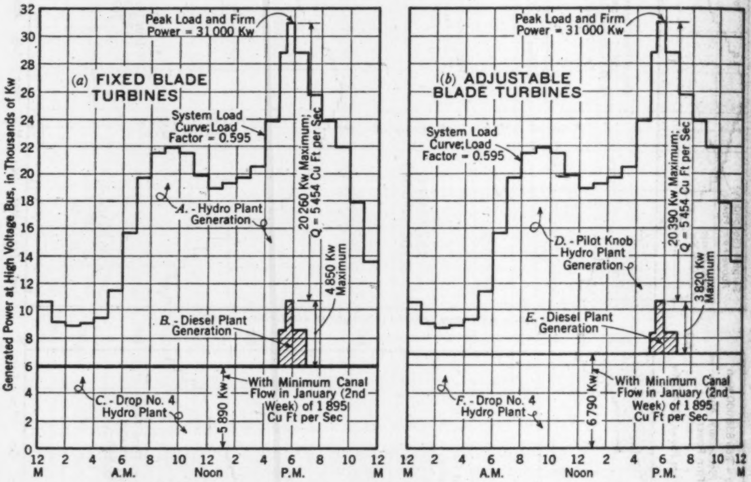


FIG. 5.—SYSTEM DAILY LOAD CURVE—USING FIXED-BLADE AND MOVABLE-BLADE TURBINES

which areas *A* to *F*, inclusive, are further described as follows:

Area (Fig. 5)	Description
A	Hydro plant generation at Pilot Knob Plant, two to ten 150-kw units installed; 295,000 kw-hr.
B	Diesel plant generation; 15,000 kw units installed; hatched area under the curve denotes 6,400 kw-hr.
C	Hydro plant generation, Drop No. 4; two 8,800-kw units installed; 141,360 kw-hr.
D	Hydro plant generation, Pilot Knob; two to ten 150-kw units installed; 275,380 kw-hr.
E	Diesel plant generation; 15,000-kw units installed; hatched area under the curve denotes 15,000 kw-hr.
F	Hydro plant generation, Drop No. 4; two 8,800-kw units installed; 162,960 kw-hr.

The maximum water use for Drop No. 4 and Pilot Knob, at peak load, equals 1,895 + 5,454 = 7,349 cu ft per sec.

The pumped storage plan offers a possible solution of the foregoing apparently inconsistent answer, as full utility of all the power will be possible. This reasoning also applies to the selection of generator capacity to be installed in a plant. The trend will be to increase the total installation materially in order to utilize the available river or canal flow as fully as possible.

INTAKES AND PENSTOCKS

The intake losses for penstocks, including trashracks and other entrance losses, vary from 0.1 ft to 1.0 ft, with 0.5 ft representing usual losses down to the turbine for medium-head plants with short penstocks. In the designs of intakes for the Wheeler power plant, situated on the Tennessee River, careful consideration was given to the shape of the intake with respect to obtaining flows normal to the line of trashrack and eliminating eddies behind the supporting structure of the trashrack. The struts were streamlined and the section was made to converge so as to obtain the same areas at the end of the struts as the minimum area of the passage and thus pinch off the formation of the eddy current tending to form at the trailing edge of the supporting struts. Under field test this intake showed a gross loss of 0.1 ft at a flow of 9,400 cu ft per sec, corresponding to the point of best efficiency of the turbine. The velocity of water through the racks was 5 ft per sec at full load, contrasted to the usual practice of providing area equivalent to a velocity of 2 ft or 3 ft per sec for the designs that have measured losses that range between 0.5 ft and 1 ft.

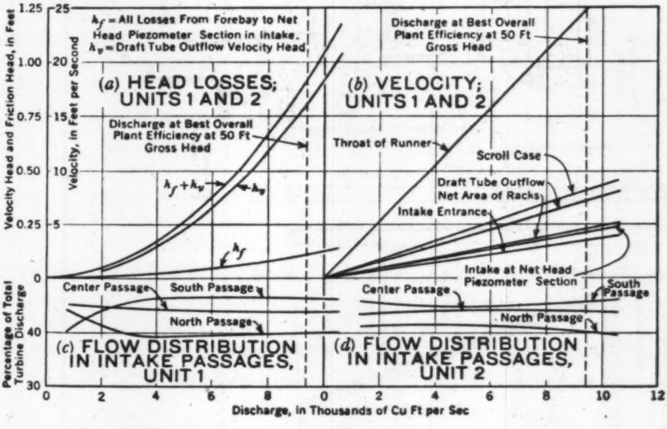


FIG. 6.—HYDRAULIC PERFORMANCE TESTS; WHEELER POWER PLANT

The field-measured characteristics for this intake are shown in Fig. 6, and a typical cross section through the plant (except an overhead 270-ton traveling crane) is shown in Fig. 7. In Fig. 6(*a*), h_f = all losses from the forebay to the

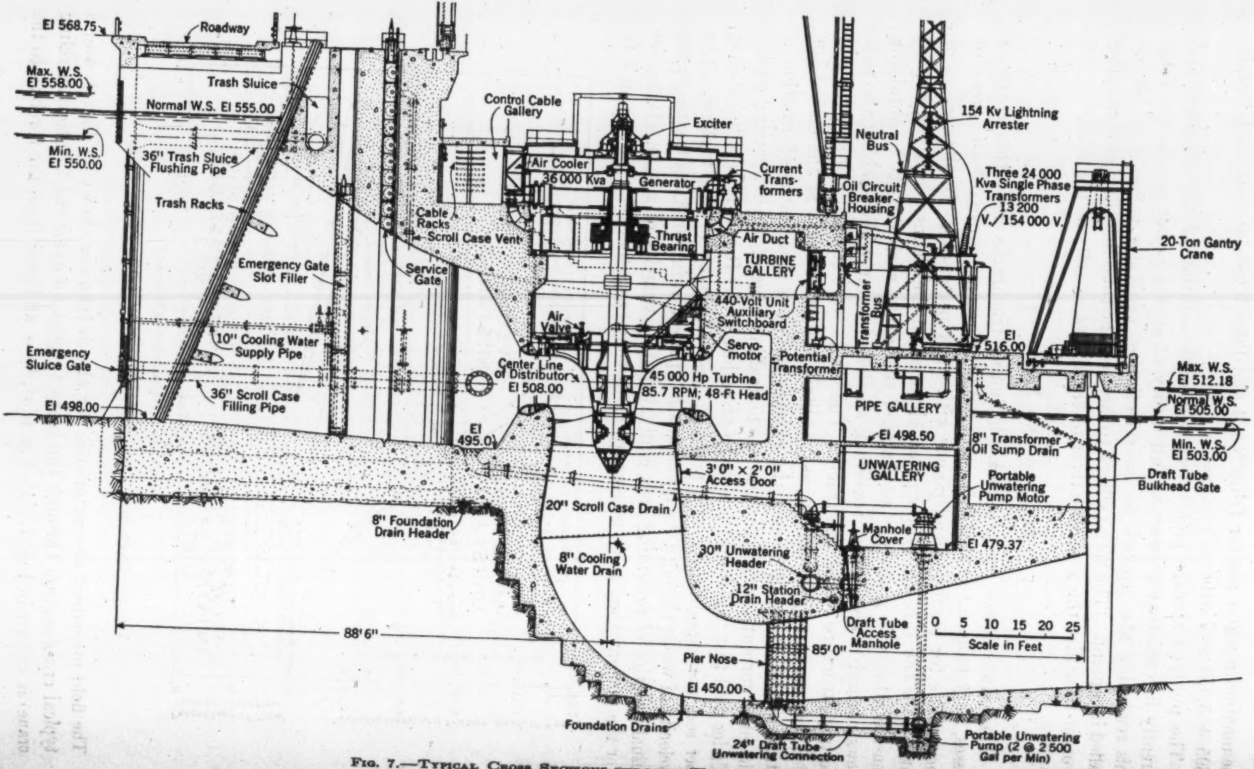

El 568.75
Roadway
Max. W.S. El 558.00
Trash Sluice
Normal W.S. El 555.00
Min. W.S. El 550.00
36" Trash Sluice Flushing Pipe
Trash Racks
Emergency Gate Slot Filler
10" Cooling Water Supply Pipe
Emergency Sluice Gate
36" Scroll Case Filling Pipe
El 498.00
Control Cable Gallery
Cable Racks
Scroll Case Vent
Service Gate
Exciter
Air Cooler
36 000 Kva
Generator
Thrust Bearing
Air Valve
Center Line of Distributor
El 508.00
El 495.0
45 000 Hp Turbine
85.7 RPM; 48-Ft Head
Servo-motor
Current Trans-formers
Air Duct
TURBINE GALLERY
440-Volt Unit Auxiliary Switchboard
Potential Transformer
Neutral Bus
Oil Circuit Breaker Housing
Transformer Bus
154 Kv Lightning Arrester
Three 24 000 Kva Single Phase Transformers 13 200 V./154 000 V.
El 516.00
20-Ton Gantry Crane
Max. W.S. El 512.18
Normal W.S. El 505.00
Min. W.S. El 503.00
PIPE GALLERY
El 498.50
8" Transformer Oil Sump Drain
UNWATERING GALLERY
Portable Unwatering Pump Motor
Draft Tube Bulkhead Gate
3'0" × 2'0" Access Door
20" Scroll Case Drain
8" Foundation Drain Header
8" Cooling Water Drain
30" Unwatering Header
Manhole Cover
El 479.37
12" Station Drain Header
Draft Tube Access Manhole
0 5 10 15 20 25
Scale in Feet
88'6"
85'0"
Pier Nose
El 450.00
Foundation Drains
24" Draft Tube Unwatering Connection
Portable Unwatering Pump (2 @ 2 500 Gal per Min)

FIG. 7.—Typical Cross Sections through Wheeler Power Plant

net-head piezometer section in the intake; and h_v = the outflow velocity head in the draft tube.

A similar intake was designed for the power plant at Drop No. 4, on the All-American Canal. This intake, complete with homologous turbine, was built and tested in the turbine manufacturer's laboratory and the total loss in head, including 80 ft of penstock, was determined as 0.18 ft, corresponding to the point of best efficiency on the turbine. Stepped up to prototype value, this head loss is approximately 0.15-ft total, including trashrack, supporting piers, entrance losses, and friction losses down to the turbine casing.

This type of large-volume, low-head intake was developed to reduce costs and improve efficiency. The cost was reduced by using smaller rack areas and increasing the apparent velocity through the trashrack structure; and the efficiency was increased by making the full area of the intake and trashrack effective. With the usual bell-mouth intake it is frequently necessary to install a false roof over the upper bell-mouth to obtain satisfactory flow conditions when current meters are being used in the intake section for measuring water. The improved intake, with trashrack symmetrical and normal to the center line of the penstocks, follows the experience gained by the use of these false roofs. Accelerating the flow and streamlining the struts seems to offer a satisfactory solution for the prevention of eddy formation behind supports placed in water passages. This principle of design has been used in various structures and has proved uniformly successful.

A type of intake structure with circular trashracks has been developed for the Grand Coulee power plant on the Columbia River in which all losses from forebay to the circular section of the penstock will not exceed 15% of the velocity head at this point. These intakes, as shown in Fig. 8, incorporate the streamlined feature of the trashrack structure and symmetrical intake normal to the center line of the conduit. The entrance to the penstock is further improved by using the rate of acceleration produced by the area of a *vena contracta* nozzle under spouting velocity. This design is predicated on the assumption that any increase or decrease in area of the approach transition from the gate section to the penstock, other than that of the *vena contracta*, will result in losses due to turbulence as the normal rate of acceleration is destroyed. These conclusions have been borne out by laboratory tests.

Where the center line of the penstock is not normal to the face of the dam, the intakes are further improved by introducing a velocity acceleration component to produce a change in direction of the flow into the penstock. This is accomplished by the construction of a velocity-vector diagram which gives the final velocity as a function of the initial velocity for a given change in direction, as shown in Fig. 9. The increased velocity required to produce the change in direction of flow is superimposed on the acceleration required by the *vena contracta* in accordance with Newton's second law of motion. The areas and velocities required for a transition of this design are shown in Fig. 10 and Table 1. The application of these principles of design results in an intake with the highest possible efficiency and uniform distribution of flow in the

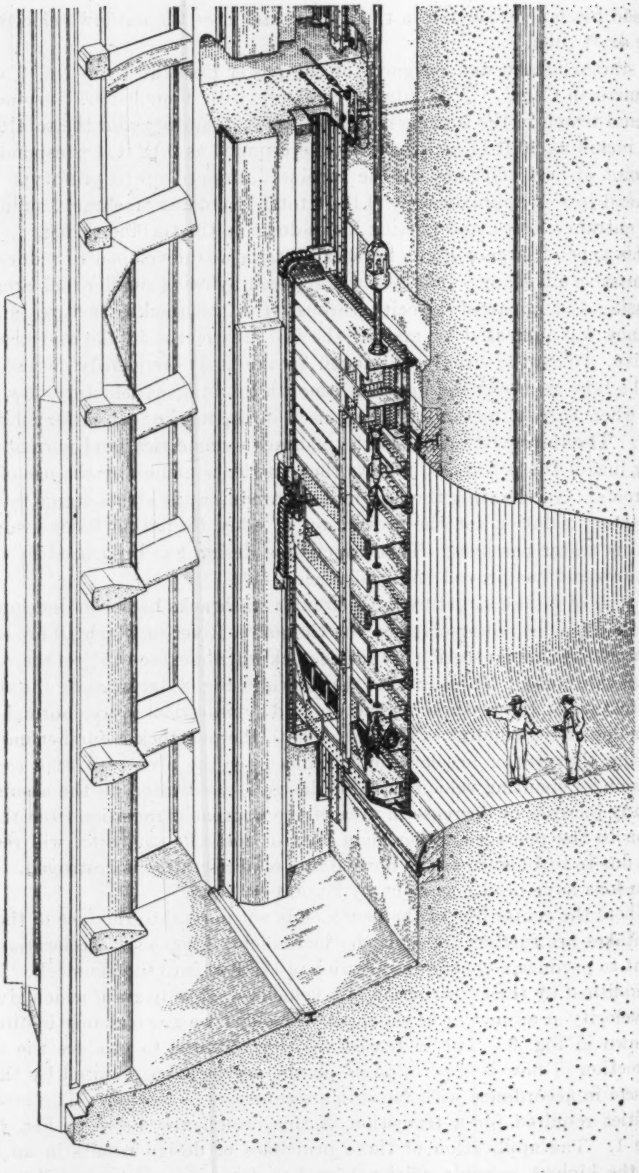
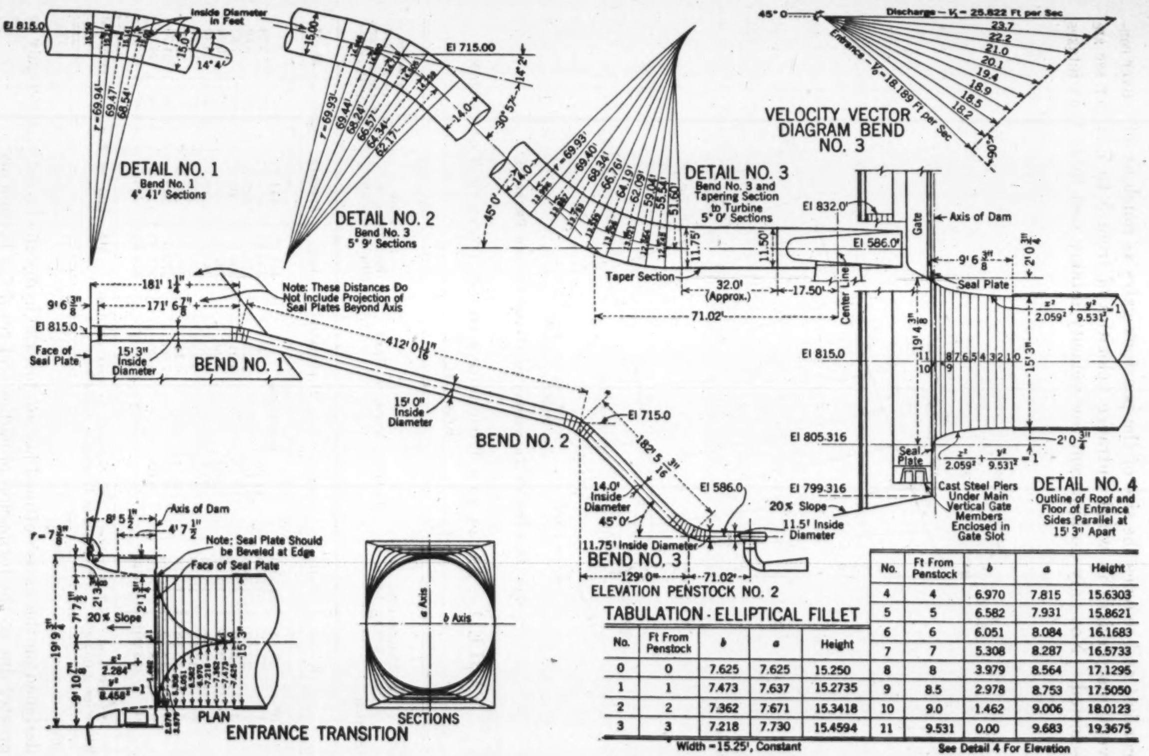


FIG. 8.—PENSTOCK INTAKE, TRASHRACK, AND GATE FOR THE GRAND COULEE POWER PLANT


DETAIL NO. 1
Bend No. 1
4° 41' Sections
El 815.0
Inside Diameter in Feet
15.0
14° 4'
r = 69.94'
69.47'
68.54'
DETAIL NO. 2
Bend No. 3
5° 9' Sections
15.00
El 715.00
14.0
30' 57'
14' 2'
r = 69.93'
69.44'
68.24'
66.57'
64.34'
62.17'
DETAIL NO. 3
Bend No. 3 and Tapering Section to Turbine
5° 0' Sections
45° 0'
14.0
r = 69.93'
69.40'
68.34'
66.76'
64.19'
62.09'
59.04'
55.54'
51.60'
11.75'
11.50'
Taper Section
32.0' (Approx.)
17.50'
71.02'
Center Line
El 832.0'
El 586.0'
Gate
Axis of Dam
VELOCITY VECTOR DIAGRAM BEND NO. 3
45° 0
Discharge – V₁ – 25.822 Ft per Sec
Entrance – V₀ – 18.189 Ft per Sec
23.7
22.2
21.0
20.1
19.4
18.9
18.5
18.2
.06
9' 6 3/8"
181' 1 1/4"
171' 6 7/8"
Note: These Distances Do Not Include Projection of Seal Plates Beyond Axis
El 815.0
Face of Seal Plate
15' 3" Inside Diameter
BEND NO. 1
412' 0 11/16"
15' 0" Inside Diameter
BEND NO. 2
El 715.0
182' 5 3/16"
14.0' Inside Diameter
45° 0'
El 586.0
11.75' Inside Diameter
11.5' Inside Diameter
BEND NO. 3
129' 0"
71.02'
ELEVATION PENSTOCK NO. 2
9' 6 3/8"
2' 0 3/4"
Seal Plate
19' 4 3/100"
El 815.0
El 805.316
El 799.316
20% Slope
8 7 6 5 4 3 2 1 0
11
10
9
15' 3"
x²/2.059² + y²/9.531² = 1
2' 0 3/4"
Cast Steel Piers Under Main Vertical Gate Members Enclosed in Gate Slot
DETAIL NO. 4
Outline of Roof and Floor of Entrance Sides Parallel at 15' 3" Apart
r = 7 3/8"
8' 5 1/2"
4' 7 1/2"
Axis of Dam
Note: Seal Plate Should be Beveled at Edge
Face of Seal Plate
2' 1 3/16"
2' 1 1/2"
7' 7 1/2"
20% Slope
19' 9 3/4"
9' 10 7/8"
x²/2.284² + y²/8.458² = 1
15' 3"
PLAN
ENTRANCE TRANSITION
a Axis
b Axis
SECTIONS

TABULATION - ELLIPTICAL FILLET

No.	Ft From Penstock	b	a	Height
0	0	7.625	7.625	15.250
1	1	7.473	7.637	15.2735
2	2	7.362	7.671	15.3418
3	3	7.218	7.730	15.4594
4	4	6.970	7.815	15.6303
5	5	6.582	7.931	15.8621
6	6	6.051	8.084	16.1683
7	7	5.308	8.287	16.5733
8	8	3.979	8.564	17.1295
9	8.5	2.978	8.753	17.5050
10	9.0	1.462	9.006	18.0123
11	9.531	0.00	9.683	19.3675

Width = 15.25', Constant

See Detail 4 For Elevation

Fig. 9.—Hydraulic Designs for Tapering Penstock

conduit and reduces the size of the penstock gates as much as 30% over conventional designs in which entrance velocities of from 5 to 7 ft per sec are selected. Not only do apparent low-velocity intakes cost more than intakes

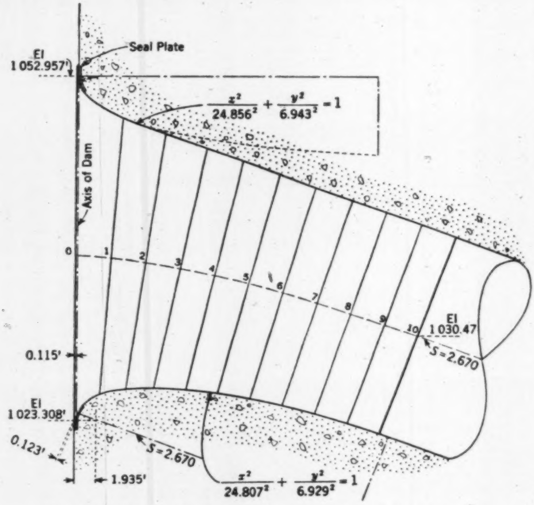


FIG. 10.—INTAKE TRANSITION OF GRAND COULEE PENSTOCK

TABLE 1.—AREAS AND DIMENSIONS OF SECTIONS IN TRANSITION (See FIG. 10)

Section	DIMENSIONS AT CENTER LINE, IN FEET		ELLIPSE AXES, IN FEET		AREA OF SECTIONS, IN SQUARE FEET		DEVELOPED DISTANCE FROM THE ORIGIN AT SEAL PLATES	
	Height	Width	Vertical	Horizontal	Jet‡	Actual§	Roof‖	Floor¶
0*	29.65	15.00	Rectangle		444.7	444.7	0	0
1	23.72	15.30	11.86	0.63	356.6	356.5	5.45	3.30
2	22.09	15.60	11.05	2.21	324.5	323.7	9.25	5.60
3	20.90	15.90	10.45	3.21	305.1	303.5	12.95	7.95
4	20.05	16.20	10.03	3.99	292.5	290.5	16.10	10.70
5	19.34	16.50	9.67	4.65	284.2	280.5	19.35	13.40
6	18.82	16.80	9.41	5.35	278.2	273.0	22.57	16.10
7	18.48	17.10	9.24	6.18	275.0	267.0	25.77	18.85
8	18.25	17.40	9.13	7.05	273.0	262.3	28.80	21.70
9	18.10	17.70	9.05	8.05	271.9	257.8	31.92	24.60
10	18.00†	18.00†	Circle		271.7	254.5	34.95	27.40

* Gate seal. † Diameter. ‡ Normal to axis of dam. § Normal to flow. ‖ Origin at Elevation 1,052.957. ¶ Origin at Elevation 1,023.308.

designed on the basis outlined herein, but the hydraulic losses may actually be greater due to the formation of eddies and resulting turbulence.

The location of the forebay apron with respect to the lower gate sill shown in Fig. 8 is determined by the velocity formulas presented in 1926 by C. W.

Harris,[3] M. Am. Soc. C. E. Referring to Fig. 11:

$$y = \frac{K'}{x^4} \quad \text{(2)}$$

In Equation (2), if $C_c = 0.6$,

$$K' = \frac{0.6^2}{4} r^4 H \quad \text{(3)}$$

and

$$y = \frac{0.09\, r^4 H}{x^4} \quad \text{(4)}$$

in which: r = radius of orifice at the wet face of the plate, and H = the head on the orifice. To orient the curve, let $x^4 = \left(x - \frac{z}{x^4}\right)^4$. By limiting conditions $z = 0.4523\, r^5$. Under conditions of spouting flow through the orifice, the pressure reduction at a distance x from the center line (Fig. 11) is expressed by

$$y = \frac{0.09\, r^4 H}{x - \left(\frac{0.4523\, r^5}{x^4}\right)^4} \quad \text{(5)}$$

In terms of velocity and penstock diameter, Equation (5) may be written:

$$v = \frac{0.125\, D^2 V}{x - \left(\frac{0.507\, D^5}{x^4}\right)^2} \quad \text{(6)}$$

in which v = velocity at a point distant x from the center line of the penstock (measured parallel to the face); D = diameter of penstock; and V = velocity of the water in the penstock. Tests conducted by the U. S. Bureau of Reclamation indicate that values determined by Equation (6) are slightly greater than those found in a model. For small values of v, as compared to V (that is, $v < 10\%\ V$), the following approximate formula can be used:

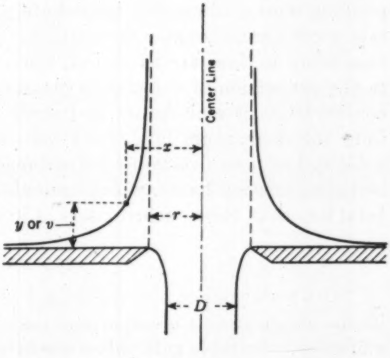


Fig. 11

$$x = 0.3531\, D \left(\frac{V}{v}\right)^{0.5} \quad \text{(7)}$$

The velocity curve is constructed in accordance with Equation (6) or Equation (7) on the basis of the head on the orifice being the velocity head in the penstock. A point is selected along the face of the inlet for the location of the apron which corresponds to the silting velocity used in the analysis of the intake.

It is assumed that the apron should not be lower than the point where silting will occur, as the rock area below this point will be ineffective. This basis of design has been verified in part by reports of the investigation of the silt level in the intake structures of the sluice-gate chamber at the power plant of the Tennessee Valley Authority at Norris Dam.

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[3] "Pressure Reduction on the Face of Orifice Plates and Weirs," by C. W. Harris, *Bulletin No. 35*, Univ. of Washington, 1926.

The application of basic principles of design to elbows and turns in the penstock is producing passages with a high degree of hydraulic efficiency. By the application of the resolution of velocities to turns in the penstock, it is possible to prevent reflected turbulence inherent when constant-area elbows and turns are used. Approximately streamlined flow is obtained by increasing the velocity around the turns. In most penstock systems, accelerating elbows can be used without obtaining too small a final pipe size, since the pressure head and water-hammer head increase progressively as the length of penstock is increased; and thus the economical diameter of the pipe decreases. A penstock system designed in accordance with this principle is shown in Fig. 9.

In designs where several decreases in size of penstock at turns, as shown in Fig. 9, are not desirable, accelerating elbows may be used by placing an expanding section of conduit immediately downstream from the elbow to maintain a constant diameter of penstock. The so-called inherent shock losses in expanding sections are minimized when the areas of the sections are favorable to the conversion of velocity to pressure head in accordance with the laws of conservation of momentum and pressure. Expanding sections, intended to fulfil the momentum laws, have been designed and tested, and the outflow velocity has been determined experimentally as 98% of the spouting velocity, including friction losses, which undoubtedly account for a greater part of the total loss than the so-called losses of impact.

GATES AND VALVES

With water power now recognized as a national resource to be developed to its maximum use, it is important that valves be designed for high hydraulic efficiency. Suitable gate valves are being developed in large sizes and for high heads incorporating passages that are straight pipe sections without hydraulic losses. Where butterfly valves appear to be the only practical solution, water passages may be so designed as to result in a loss of head through the valve of not more than 10% of the velocity head in the pipes. Losses through the 120-in. butterfly valves at the power plant at Boulder Dam are shown in Table 2 and Fig. 12. The low losses for this valve are due to the small discharge diameter as compared with the nominal diameter, resulting in accelerated flow that suppresses eddy formation downstream from the valve disk. Usually these losses vary from 20% to 30% of the pipe velocity head when only the mechanical features of the valve are considered.

One of the most important points in gate-valve design has been determined as the hydraulic downthrust on top of the gate leaf when the valve is being closed under spouting velocity conditions. These downward forces have been found to exceed the total weight of the gate leaf by several hundred per cent; thus, a valve leaf hoist designed to lift a load of 100,000 lb would be required to overcome a maximum load of 200,000 lb due to the hydraulic thrust alone. This condition can be relieved greatly by preventing full penstock pressure from developing on the top of the gate leaf where the valve is placed in the penstock line. This may be accomplished by making close clearances on the upstream face of the valve leaf where high pressures are developed, and liberal clearances on the downstream faces where the low pressure region exists.

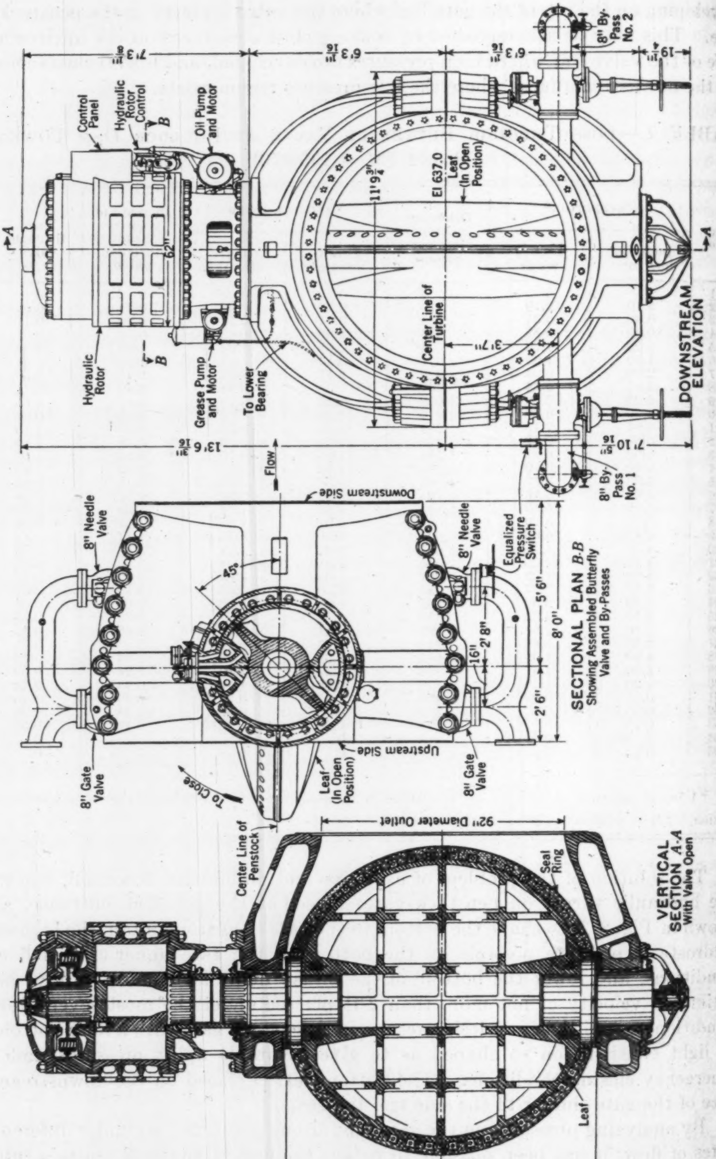
TABLE 2.—LOSS THROUGH BUTTERFLY VALVE AT BOULDER DAM POWER PLANT ($M = 135{,}670$)

Run No.	Deflection*		Discharge in cubic feet per second	Velocity Head		Increase in velocity head through valve	Loss through valve
	Inches of mercury	Feet of water		Down-stream†	Up-stream‡		
1	0	0	4	0	0	0	0
2	0.10	0.10	116	0.10	0.05	0.05	0.05
3	0.30	0.31	234	0.41	0.21	0.20	0.11
4	0.50	0.52	375	1.06	0.55	0.51	0.01
5	1.20	1.26	524	2.07	1.07	1.00	0.26
6	1.90	1.99	677	3.46	1.78	1.68	0.31
7	2.90	3.04	823	5.11	2.63	2.48	0.56
8	3.95	4.14	967	7.06	3.64	3.42	0.72
9	5.05	5.29	1,099	9.12	4.70	4.42	0.87
10	6.09	6.38	1,219	11.22	5.78	5.44	0.94
11	6.81	7.13	1,281	12.39	6.39	6.00	1.13
12	6.80	7.12	1,276	12.30	6.34	5.96	1.16
13	6.60	6.91	1,258	11.95	6.15	5.80	1.11
14	6.40	6.70	1,239	11.60	5.97	5.63	1.07
15	5.80	6.07	1,183	10.58	5.45	5.13	0.94
16	5.60	5.86	1,165	10.25	5.28	4.97	0.89
17	5.30	5.55	1,127	9.60	4.94	4.66	0.89
18	5.05	5.29	1,092	9.01	4.64	4.37	0.92
19	4.80	5.03	1,067	8.60	4.43	4.17	0.86
20	4.50	4.71	1,031	8.02	4.14	3.88	0.83
21	4.15	4.35	1,006	7.64	3.94	3.70	0.65
22	3.90	4.08	967	7.06	3.64	3.42	0.66
23	3.60	3.77	928	6.50	3.35	3.15	0.62
24	3.40	3.56	887	5.94	3.06	2.88	0.68
25	3.10	3.25	859	5.57	2.87	2.70	0.55
26	2.80	2.93	823	5.11	2.63	2.48	0.45
27	2.35	2.46	743	4.17	2.14	2.03	0.43
28	1.90	1.99	669	3.38	1.74	1.64	0.35
29	1.50	1.57	583	2.57	1.32	1.25	0.32
30	1.20	1.26	519	2.04	1.10	0.94	0.32
31	0.85	0.89	449	1.52	0.78	0.74	0.15
32	0.40	0.42	305	0.70	0.36	0.34	0.08
33	0.10	0.10	173	0.23	0.12	0.11	−0.01
34	0	0	4	0	0	0	0
35	0	0	4	0	0	0	0

* 1 in. of mercury $= \frac{13.5670 - 1}{12} = 1.04725$ ft of water. † $h_v = 0.00000755\,Q^2$ at downstream section. ‡ $h_r = 0.00000389\,Q^2$ at upstream section.

The solution of the problem of excessive and prohibitive downpull, due to the hydraulic thrust on penstock gates placed at the penstock entrance, as shown in Fig. 8, is to shape the bottom to the gate so as to maintain the highest hydrostatic pressure possible on the bottom of the gate under critical flow conditions, and lower the bottom of the intake flume an amount to give an estimated velocity of not more than 2 ft per sec under the most severe flow conditions. This basis of design results in a penstock gate with a false bottom of light construction so shaped as to give favorable uplift pressures under emergency closing conditions. The bottom seal is placed on the downstream face of the gate similar to the side and top seals.

By analyzing pressures on the gate for various positions and under different rates of flow, it has been possible to reduce the cost of the gate hoists a sub-



DOWNSTREAM ELEVATION

SECTIONAL PLAN B-B
Showing Assembled Butterfly Valve and By-Passes

VERTICAL SECTION A-A
With Valve Open

Control Panel
Hydraulic Rotor Control
Oil Pump and Motor
Hydraulic Rotor
Grease Pump and Motor
To Lower Bearing
62"
B
A
El 637.0
Leaf (In Open Position)
11' 9¾"
Center Line of Turbine
3' 7"
7' 3 $\frac{3}{16}$"
6' 3 $\frac{3}{16}$"
6' 3 $\frac{3}{16}$"
13' 6 $\frac{3}{16}$"
7' 10 $\frac{3}{16}$"
19¼"
8" By-Pass No. 2
8" By-Pass No. 1
Flow
Downstream Side
Upstream Side
8" Needle Valve
8" Needle Valve
8" Gate Valve
8" Gate Valve
Equalized Pressure Switch
45°
16"
2' 8"
5' 6"
8' 0"
2' 6"
Leaf (In Open Position)
To Close
Center Line of Penstock
92" Diameter Outlet
Seal Ring
Leaf
117 $\frac{9}{16}$" Diameter Inlet

FIG. 12.—BUTTERFLY VALVE (120-IN.) AT BOULDER DAM (SEE TABLE 2 FOR THE VELOCITY HEAD LOSS FOR VARIOUS RATES OF FLOW)

stantial amount and to increase the hydraulic efficiency of the intake. In addition to these advantages, trouble due to debris collecting in the recess slot at the bottom seal has been eliminated. By the use of a vertical face at the gate entrance for a distance of from 6 ft to 7 ft below the intake, it is unlikely that any form of material will collect on the lower seats. All of this fits into the general scheme of developing intakes that utilize higher velocities, obtain better efficiency, and are constructed at lower cost.

SCROLL CASES

It is believed that there are possibilities in the development of scroll cases with appreciably higher velocities than those normally employed which is in agreement with the results obtained in the series of pump tests previously mentioned. Such cases may not require movable gates or guide vanes, the power of the unit being controlled by a single valve placed at the entrance of the casing similar to the Reiffenstein turbine.[4] In the present stage of development the single-gate turbine is suitable principally for small units, as the control features are inferior to the present type. Furthermore, the unbalanced side thrust from the runner is high, requiring a large shaft. The relatively large field clearances and the tangential velocity component are too high for use in connection with the propeller turbine. Rapid progress is reported toward a solution of these difficulties by the study of the characteristics of the forced spiral vortex and the application of these principles in the design.

Several low-head plants have been built with improved scroll cases, including those at Wheeler Dam, Bonneville Dam, Drop No. 4, and other plants. The casings at Wheeler Dam and Drop No. 4 have three penstock intakes coming into one scroll-case chamber, the water seeking its direction into the casing as required. Sectional plan of the Drop No. 4 casing is shown in Fig. 13. The Bonneville casing employs an island to form two separate channels in one third of the casing. Results of field and laboratory tests indicate that the distribution in each of the three passages at Bonneville was substantially equal. The distribution at Wheeler and Drop No. 4 varied between 30% and 40% for the three separate penstocks. The development work on these designs consistently indicated that better guiding of the water at increased velocity tended to produce better performance. This fact was demonstrated at Wheeler Dam by an increase in horsepower of the turbine throughout the full range of gate opening when the areas in the small part of the casings were restricted by contracting the floor and roof of the casing to produce better guiding of the water into the turbine.

As an example of the results of careful attention to basic hydraulic principles in design, it was possible to remove the trashrack structures, complete intake with penstock, and the scroll case from the model turbine used in the acceptance tests of the Drop No. 4 turbine, and not lose any over-all efficiency or horsepower capacity of the turbine. In fact the detail characteristics of the gate-opening speed curves were slightly better for the test when the power-plant conduits were in place than were those obtained for the open-flume conditions. The reason for these apparently inconsistent results is obviously better distribution of flow of water into the turbine guide case with the scroll case in place than with the turbine tested under open-flume conditions. Thus, the

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[4] *Wasserkraft und Wasserwirtschaft*, Heft 11–12 (33, Jahrig. 1938), pp. 144 and 145.

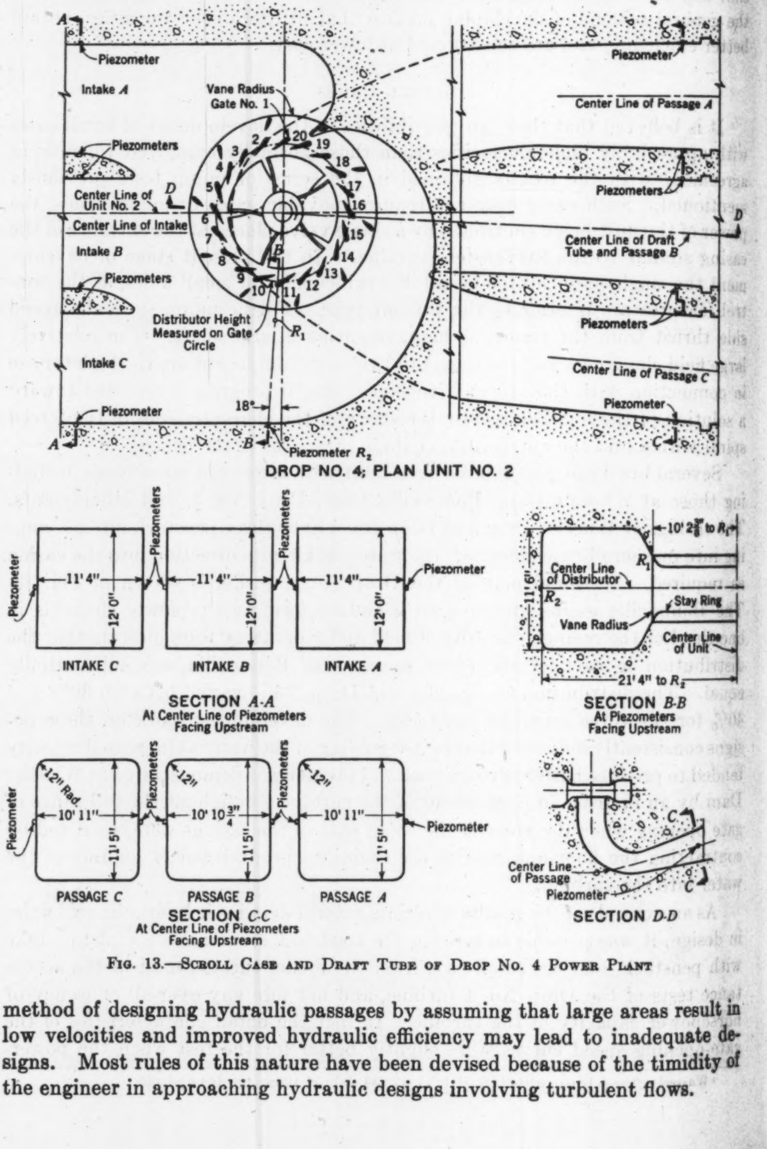
A
Piezometer
Intake A
Vane Radius Gate No. 1
Piezometers
Center Line of Unit No. 2
D
Center Line of Intake
Intake B
Piezometers
Distributor Height Measured on Gate Circle
R1
Intake C
Piezometer
18°
A
B
Piezometer R2
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
B
C
Piezometer
Center Line of Passage A
Piezometers
D
Center Line of Draft Tube and Passage B
Piezometers
Center Line of Passage C
Piezometer
C
DROP NO. 4; PLAN UNIT NO. 2
Piezometer
11' 4"
12' 0"
Piezometers
11' 4"
12' 0"
Piezometers
11' 4"
12' 0"
Piezometer
INTAKE C
INTAKE B
INTAKE A
SECTION A-A
At Center Line of Piezometers Facing Upstream
10' 2⅜" to R1
R1
Center Line of Distributor
R2
11' 6"
Stay Ring
Vane Radius
Center Line of Unit
21' 4" to R2
SECTION B-B
At Piezometers Facing Upstream
Piezometer
12" Rad.
10' 11"
11' 5"
Piezometers
12"
10' 10¾"
11' 5"
Piezometers
12"
10' 11"
11' 5"
Piezometer
PASSAGE C
PASSAGE B
PASSAGE A
SECTION C-C
At Center Line of Piezometers Facing Upstream
C
C
Center Line of Passage
Piezometer
SECTION D-D

Fig. 13.—Scroll Case and Draft Tube of Drop No. 4 Power Plant

method of designing hydraulic passages by assuming that large areas result in low velocities and improved hydraulic efficiency may lead to inadequate designs. Most rules of this nature have been devised because of the timidity of the engineer in approaching hydraulic designs involving turbulent flows.

DRAFT TUBES

The trend of draft-tube design in recent years has been toward the elbow, as the concentric type of tube has proved to be expensive, with no advantage in efficiency or power over the elbow tube. A strong tendency is now apparent for American engineers to take up the problem of elbow draft-tube design incorporating the horizontal splitter, although the European engineers who developed this type of tube are discarding it for the plain elbow. This change in opinion may be the result of the attempt to lower excavation costs in America and the attempt to obtain better efficiency in the European installations. The energy of the engineers in the United States will be directed toward both of these objectives as more of the characteristics of the splitters are better understood.

EXPERIMENTAL INVESTIGATIONS

In low-head, large-volume turbine installations, where field measurements of water are exceedingly difficult, acceptance of performance is being based on laboratory tests of strictly homologous model units, incorporating, as completely as possible, all water passages of the prototype that affect the performance of the turbine. This method of turbine acceptance offers advantages over a field test in the respect that if the tests indicate that changes in design are desirable, the necessary development work can be performed and incorporated in the final design. Where desirable changes in design are discovered during field tests on the prototype, it is usually too expensive, or it is impracticable, to make such changes in the plant.

As an alternate to model tests in the laboratory, it is sometimes feasible to base the turbine acceptance tests on a small station-service unit which is similar to the main unit. In effect, this is the use of a model of intermediate size and offers the advantage of both the field test and model test; but it is not favorable from the standpoint of development work during tests.

The field performance of a turbine that has been accepted on the basis of laboratory tests on homologous units can be estimated by suitable power step-up methods which evaluate the increase or decrease in horsepower between model and prototype, as a measure of the efficiency of the field unit. Calibration of the deflection of the differential pressure taps, placed in the model of a scroll case, as related to the flow through the turbine in cubic feet per second, can be used to estimate the flow in the prototype by placing similar piezometer connections in the scroll case.

Fig. 13 represents a typical installation of piezometer taps designed to coordinate pressure in the penstock, scroll case, and draft tube for the comparison of model and prototype hydraulic performance. By placing a V-tube manometer across the pressure taps in the lower section of the penstock at section *AA* and at the discharge orifice of the draft tube at section *CC* the true net head acting on the turbine is obtained and also the coefficient of the turbine as a meter. A second V-tube manometer to measure the differential pressure in the scroll case designated as R1–R2 at section *BB* affords a second independent comparison of flow in the model and prototype. Thus it is possible to obtain a comparison of power and discharge between model and prototype.

At the U. S. Engineers' hydraulic laboratory at Portland, Ore., studies of draft-tube shapes were made with pyralin models constructed to a scale of 1 : 46—that is, with a 6-in. throat diameter, first alone and later with model intakes and runners. Several alternatives of the elbow type were studied. The most valuable result from these experiments was the establishment of the fact that a horizontal spreader improved the stability and the efficiency of all draft tubes studied, both alone and with scroll and runner attached. These experiments were further verified at the laboratory of the turbine manufacturer, using an homologous turbine runner 16 in. in diameter.

Extensive model tests of different types of intakes on a 1 : 46 scale were made, using wood with pyralin windows. Visual methods of analysis were used with ribbons and sawdust to show the direction of flow. Piezometers were placed in the different intake passages to obtain equal head losses and equal water distribution among the passages. Motion pictures were taken and these were studied to improve the flow conditions within the water passages. By these means of investigation, very close to equal distribution of flow around the speed ring was obtained. The tests and studies resulted in the selection of an intake with long division piers between the water channels and with islands in one of the three channels to distribute the water proportionately around the wheel casings. A constant velocity scroll gave the best results, although both accelerating and decelerating scrolls were investigated.

ACKNOWLEDGMENTS

The following members of the Committee of the Power Division, on Progress in Power Plant Design (through its Sub-Committee on Progress in Prime Mover Design and Efficiency), contributed valuable assistance in the preparation of this paper: Paul L. Heslop, M. Am. Soc. C. E., LeRoy M. Davis, Assoc. M. Am. Soc. C. E., Prof. Ernest Brown, J. F. Davenport, and R. V. Terry.

MODEL AND PROTOTYPE TESTS

BY L. M. DAVIS,[5] ASSOC. M. AM. SOC. C. E.

SYNOPSIS

The development of the methods of testing turbine models is described in this paper. The writer deals particularly with the more recent progress that has been made in determining cavitation characteristics. The interpretation of the laboratory results are important not only for designing the proper setting of the prototype to avoid cavitation, but also for predicting the power that can be expected. The application of the Moody formula for this purpose is discussed. Three outstanding methods used in the United States for measuring the quantity of water taken by even the largest hydraulic units can be depended upon for reliable results, and failure to make acceptance tests may retard progress materially in the design of future installations. In conclusion the collection of data by which the step-up formulas can be verified is emphasized as being of utmost importance.

INTRODUCTION

Water wheels were in use for many centuries with only a few improvements on the original crude designs. It was not until 1827 that the modern turbine was invented by Fourneyron. About twenty years later the Francis turbine was developed by means of careful laboratory tests conducted by the late James B. Francis,[6] Hon. M. Am. Soc. C. E. By the beginning of the twentieth century, Francis turbines were available for heads up to several hundred feet, with efficiencies of about 80%. The increasing popularity of electric power tremendously increased the demand for turbines. Numerous concerns went into the business of manufacturing turbines, and many of them built turbine testing laboratories as a means of improving their product to meet competition. By 1915 the art of designing Francis turbines had advanced so that efficiencies in excess of 90% were common.

Since that date the improvement in turbine efficiency has been slight, but efficient designs have been developed for heads up to 1,000 ft. Development of the impulse wheel has followed that of the Francis turbine so that their efficiency is now only slightly less than 90%.

In the early days of turbine design it was considered impossible to have the peripheral speed of an efficient turbine exceed the spouting velocity of the water. Low heads resulted in low spouting velocities, which limited the peripheral speed. Since an increase in runner diameter meant a reduction in rotative speed and an increase in generator size for the same peripheral speed, it was common practice to connect several small turbines to one generator to obtain the desired power without sacrificing rotative speed. In some instances as many as six turbines were connected to one generator.

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[5] Hydr. Test Engr., Pennsylvania Water & Power Co., Holtwood, Pa.

[6] *Encyclopædia Britannica*, 14th Edition, Vol. 22, p. 580.

It was soon discovered that the ratio of peripheral velocity of the turbine to the spouting velocity of the water (a ratio known to hydraulic engineers as ϕ) could exceed unity without a sacrifice in efficiency.

DEVELOPMENT OF PROPELLER TURBINE

In order to reduce generator costs to a minimum, the turbine manufacturers attempted to design high-speed turbines for low-head developments. This trend led to the introduction of the propeller turbine, which has a high specific speed.

Specific speed may be defined as the revolutions per minute at which a runner would rotate if it were so reduced in proportion that it would develop 1 hp under a 1-ft head. In runners with high specific speeds the relative velocity between the water and runner is greater than for runners with low specific speeds. It is evident from Fig. 14 that in attaining high specific speeds there was a considerable sacrifice in efficiency at part loads.

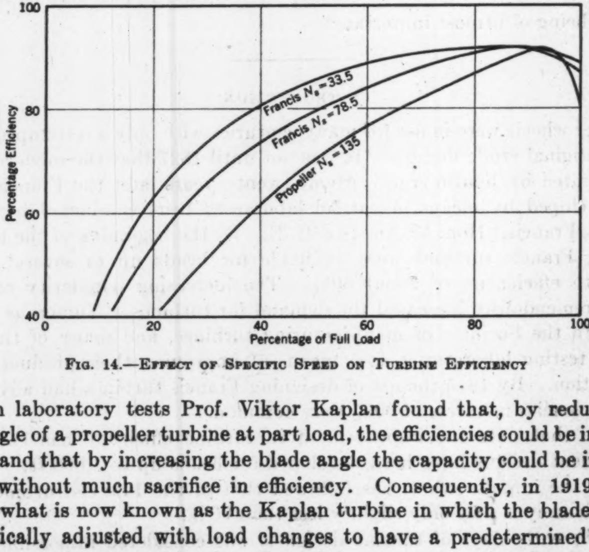

FIG. 14.—EFFECT OF SPECIFIC SPEED ON TURBINE EFFICIENCY

From laboratory tests Prof. Viktor Kaplan found that, by reducing the blade angle of a propeller turbine at part load, the efficiencies could be improved greatly, and that by increasing the blade angle the capacity could be increased greatly without much sacrifice in efficiency. Consequently, in 1919, he developed what is now known as the Kaplan turbine in which the blade angle is automatically adjusted with load changes to have a predetermined relation to the gate opening, resulting in a flat efficiency curve over most of the operating range, as shown in Fig. 15.

Since about 1914 a turbine testing laboratory capable of determining the efficiency characteristics of carefully made models of turbines with their settings, draft tube, intake, and scroll case has been considered an indispensable part of a turbine manufacturer's equipment. It seems unnecessary in this

paper to include a description of such a laboratory. In 1938 Mr. L. J. Hooper gave an excellent description[7] of most of the turbine testing laboratories in the United States.

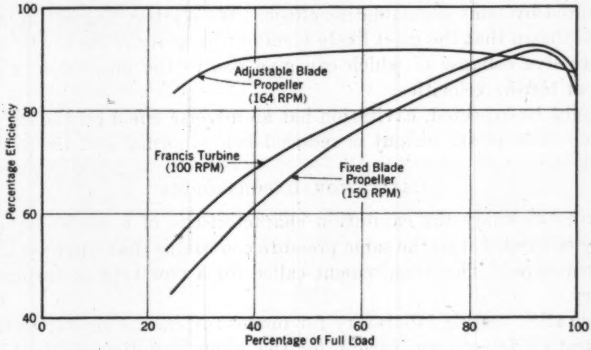

FIG. 15.—COMPARISON OF EFFICIENCY CURVES OF THREE 9,500 HP TURBINES AT 38-FT HEAD

CAVITATION AS A VITAL PROBLEM

Increasing the specific speed of turbines led to very high relative velocities between the turbine blades and the water, as well as high actual velocities, with a corresponding conversion of pressure head into velocity, which in turn resulted in low pressures in the turbine. This condition has been conducive to a phenomenon known as cavitation, which has become a vital problem.[8]

Cavitation is not new to hydraulic engineers. Some of the early Francis turbines were subject to serious cavitation caused by faulty design or to locating the turbine too far above the tailwater.

As the name implies, cavitation involves the formation of cavities in the liquid. These cavities are made possible when the pressure is reduced to the vapor tension. When this occurs, the liquid "cold boils," forming vapor which fills the cavity. The cavities are carried along with the flowing water; and when the pressure increases above the vapor pressure, the vapor turns to liquid and the cavities suddenly collapse. This violent collapse creates high water-hammer pressures concentrated on a very limited area. Striking upon the boundary walls, these blows are undoubtedly repeated at a high frequency, finally resulting in a failure of the boundary material known as pitting.

The possibility that these blows are severe may be appreciated if one considers that the collapse of a spherical void in an inelastic medium, theoretically, will cause an infinite pressure at the moment of complete collapse. The elasticity of the water imposes very definite limitations which are still further reduced by the presence of non-condensable gases such as air. Con-

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[7] "Representative Hydraulic Laboratories in the United States and Canada," by L. J. Hooper, *Journal*, Boston Soc. of Civ. Engrs., January, 1938.

[8] "Vortrag auf der Weltkroftkonferenz," by D. Thoma, Germany, 1924; also "Investigations into Causes of Corrosion or Erosion of Propellers," by Sir Charles A. Parsons and Stanley S. Cook, *Engineering*, April 18, 1919.

sidering the elasticity of water, it is difficult to believe that the fatigue limits of the materials used can be exceeded. The fact that pitting actually can be produced in any material so far studied leads to the supposition that the pressure is generated not by the simple falling in of the walls of a sphere, but is augmented by some accumulative effect. W. Watters Pagon,[9] M. Am. Soc. C. E., has shown that the most likely form of the cavity is the core of a vortex, the progressive collapse of which can give rise to the projection of particles of water at terrific velocities.

As might be expected, cavitation has an adverse effect on the efficiency of a turbine. The power output is reduced below normal and the discharge is increased.

CAVITATION LABORATORIES

In order to study the cavitation characteristics of a model turbine, it is necessary to subject it to the same pressure conditions that will be encountered in the prototype. This requirement called for a new type of turbine testing laboratory.

A cavitation testing laboratory for model turbines is similar to the usual turbine testing laboratory, except for the wider variation of the headwater and tailwater levels and for the higher test heads. In order to reproduce all possible head conditions to which a turbine may be subjected, it is necessary to be able to vary the headwater and tailwater levels over a wide range, thus varying the draft head and the pressure conditions on the turbine. When first attacking the problems of determining the cavitation characteristics of a turbine design, it appeared necessary to test the model under the same head conditions which prevail in the field. Experience has now shown that this is not necessary. In the case of the Holtwood (Pa.) laboratory where cavitation data were taken for heads varying from 45 ft to 58 ft, there was no noticeable effect due to head variation. However, Prof. W. Spannhake conducted similar tests in the cavitation laboratory at Karlsruhe, Germany, under heads from 1 m to 10 m (3.3 ft to 32.8 ft) and found a decided influence of head at the lower values. This effect seemed to be completely "ironed out" by the time the 10-m head was reached. Of course, in order to obtain the desired draft head when the total head is low, it is necessary to reduce the pressure on the entire system.

Since several headwater and tailwater combinations result in the same pressure conditions in the runner, it is essential to have a cavitation coefficient which will indicate the true pressure conditions on the runner for any possible combinations. Through his outstanding work[10] at Munich, Germany, Prof. D. Thoma originated the commonly accepted cavitation coefficient, Σ. The formula for Σ is:

$$\Sigma = \frac{H_b - H_s - H_v}{H_t} \qquad (8)$$

in which H_b = barometric pressure in feet of water; H_s = draft head, in feet; H_v = aqueous vapor pressure, in feet of water; and H_t = total head, in feet.

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[9] "Cavitation and Erosion Investigated as a Problem in Fuid Mechanics," by W. W. Pagon, presented at the Annual Meeting of the A. S. M. E., 1935.

[10] Described in a Talk by Prof. Thoma at the Fall Meeting of the Society, October 10, 1929. For a brief abstract of the Paper see *Proceedings*, Am. Soc. C. E., December, 1929, p. 2561.

The first cavitation testing laboratory for turbine models in the United States was built in 1930 by the Pennsylvania Water and Power Company at Holtwood for use in designing the 42,000-hp Kaplan turbines for the Safe Harbor Development. The Shawinigan Water and Power Company of Montreal, Que., Canada, built their cavitation turbine testing laboratory prior to 1930, but it was in use on an extended testing program during the time the Safe Harbor tests were conducted. There were also certain head limitations which did not fulfil the requirements for the Safe Harbor tests. The layout of the Holtwood laboratory and the procedure of conducting cavitation tests was described in two papers published[11] in 1934 and 1935.

There are three other laboratories in the United States capable of determining the cavitation characteristics of a model turbine. The test procedure is essentially the same in all of them. Three of the four laboratories are equipped to test models of the complete unit, including intake passages, scroll case, wheel setting, turbine, and draft tube. A complete model of all water passages is usually considered highly desirable since disturbances from upstream of the turbine may have an effect on the cavitation characteristics.

In the early days of cavitation testing of model turbines there was considerable controversy as to the best means of determining the point at which cavitation begins as the draft head is progressively increased. Most early laboratories were equipped with glass windows in the draft tube for visual observation. Although it is extremely interesting to observe the turbine under various operating conditions, the visual method of determining the starting point of cavitation was found to be unreliable. Not only would two people place a different interpretation on what they observed, but it is impossible for the same person to compare visual observations accurately from day to day.

At present the commonly accepted method of determining the start of cavitation is by quantitative measurement, making use of the fact that cavitation reduces power and increases discharge, with a resulting drop in efficiency. Fig. 16(a) shows the data obtained from a cavitation test at three gate openings and at a constant value of ϕ. This test was at one blade setting of a model of a Kaplan turbine. It will be noted that efficiency, unit horsepower, and unit discharge are all plotted against Σ.

It should be stated that Σ is used in two ways: (1) In reference to the plant and (2) in reference to the turbine. Plant sigma is the effective sigma as computed for the installed unit. The value of plant sigma will vary with headwater and tailwater elevations, barometric pressure, and vapor pressure of the water, which in turn varies with water temperature. Critical sigma is the value at which cavitation begins in a turbine for a given set of conditions of gate opening, blade angle (for Kaplan or adjustable-blade propeller turbines), speed, and head. The purpose of the cavitation laboratory is to determine the critical sigma values for various operating conditions. It follows logically that if the plant sigma should be less, at any time, than the critical sigma, cavitation could be expected.

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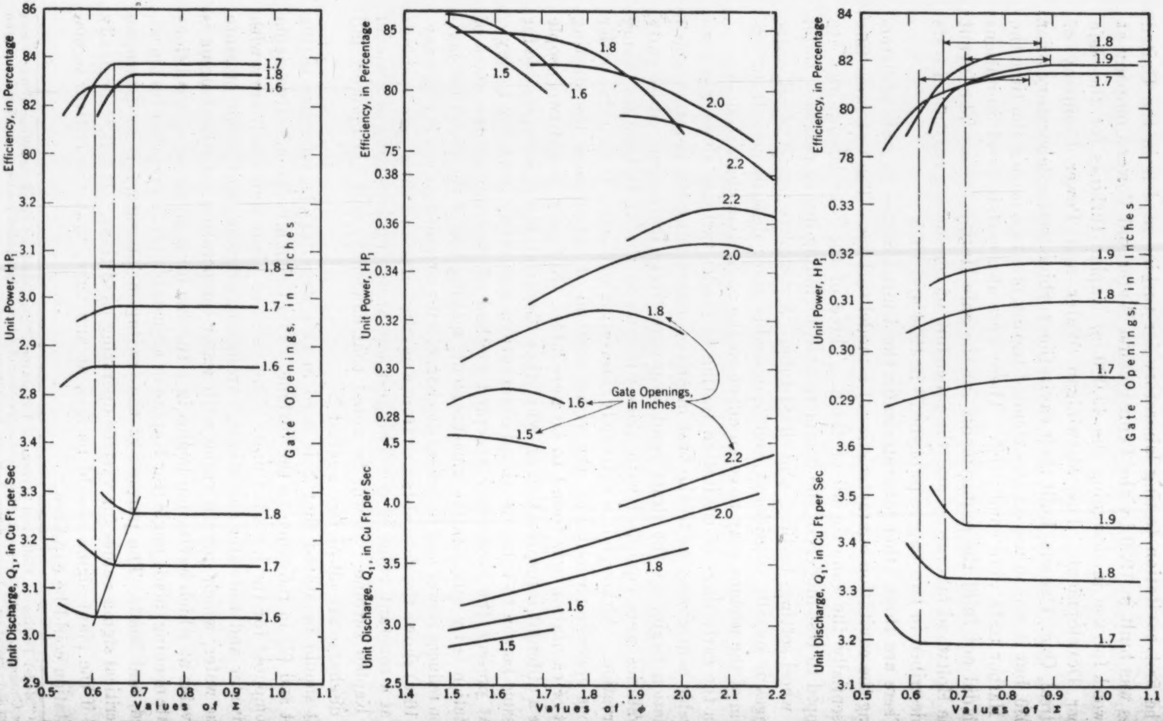
[11] "Model Testing at Holtwood Hydraulic Laboratory," published by the Pennsylvania Water and Power Company, May, 1934; and, "Cavitation Testing of Model Hydraulic Turbines and Its Bearing on Design and Operation," *Transactions*, A. S. M. E., November, 1935.



(*a*) **Cavitation Test (Σ); Properly Designed Propeller; Runner 2.1; $\phi = 1.635$**

(*b*) **Efficiency Test (ϕ); Runner 2.1; Head, 36 Ft; $\Sigma = 1.22$; Alternate Gate Curves Omitted**

(*c*) **Cavitation Test (Σ); Poorly Designed Propeller; Runner 1.2; $\phi = 1.686$**

FIG. 16.—TYPICAL RESULTS ON A PROPELLER TURBINE (BLADE ANGLE, 26°; GATE POSITIONS AS SHOWN)

To determine the cavitation characteristics of a fixed-blade propeller turbine, it is necessary to run tests at three or four values of ϕ covering the operating range of heads and for various gate settings from the maximum down to a low enough gate setting so that the critical sigma is well below the lowest value of plant sigma. In the case of a Kaplan or adjustable-blade propeller turbine, this procedure would require much unnecessary work since such a turbine is operated so that the most efficient relation between gate opening and blade angle is maintained at all times. In order to secure the information in the operating range only, efficiency tests are made over the entire operating range of ϕ-values for each blade setting of the model (see Fig. 16(*b*)). From these data, the three gate settings which bracket the peak efficiency at the value of ϕ in question are determined and cavitation tests are made on them as shown in Fig. 16(*a*). It is the practice in the Holtwood laboratory to use the break in the unit-discharge-sigma curve in arriving at the critical sigma. If the efficiency and unit-power curves break at a higher value of sigma than the unit-discharge curve, this higher value is used with the unit discharge obtained at that sigma. Cavitation tests are made at three or four values of ϕ so that the data may be presented as shown in Fig. 17. It is an easy matter to cross-plot and locate intermediate ϕ-curves.

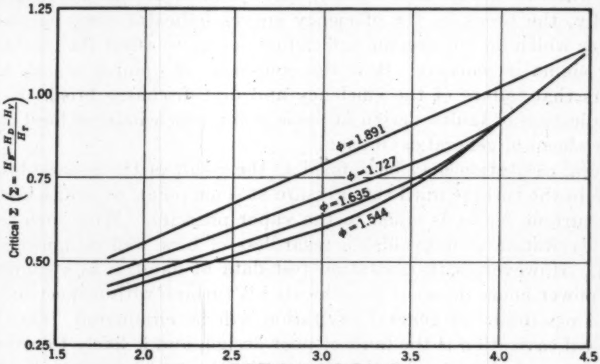


FIG. 17.—CRITICAL VALUES OF Σ VERSUS UNIT DISCHARGE FOR VARIOUS VALUES OF ϕ

Applying the data in Fig. 17 to a specific development, it is possible to determine the proper elevation of the turbine with reference to tailwater levels and also to determine the safe load that can be carried at heads other than the rated head. In arriving at the proper turbine setting after the turbine diameter, rotative speed, and maximum output at rated head have been decided upon, it is only necessary to determine the unit discharge corresponding to the maximum output at rated head and apply this value in Fig. 17 to obtain the critical Σ-value for the value of ϕ corresponding to rated head. Using this value of critical sigma in the sigma formula, the allowable draft head may be computed. It is common practice among turbine manufacturers to insist that the turbine be set a certain distance lower than indicated by this computation (1.5 ft in the case of Safe Harbor) as a margin of safety.

After the turbine setting has been determined, the plant sigma values can be computed for various headwater and tailwater conditions; and, by the use of Fig. 17, the allowable unit discharge is obtained from which the allowable power output is computed. For the convenience of operators, these data can be presented in a chart of constant power lines with headwater elevations as abscissas and tailwater elevations as ordinates.

In most developments it is of economic importance to locate the runner at as high an elevation as possible so as to keep the excavation cost at a minimum. Therefore, some idea of the importance of an accurate determination of the critical sigma of a turbine may be understood when it is realized that a variation in draft head of 1 ft at Safe Harbor affects the allowable power output 3.5% at normal head.

Fig. 16(*c*) presents data similar to those in Fig. 16(*a*), but for an entirely different turbine model. It will be noted that in Fig. 16(*c*) there is a wide discrepancy between the value of critical sigma, as indicated by the break in the efficiency curves and that indicated by the break in the unit-discharge curves, whereas in Fig. 16(*a*) the breaks occurred at the same sigma. Undoubtedly, the break in the efficiency curves indicates some cavitation disturbances which do not become sufficiently severe to affect the discharge until a lower sigma is reached. It is the consensus of opinion among hydraulic engineers that failure of the efficiency and unit-discharge breaks to coincide is an indication of faulty design at some point which induces local cavitation to occur ahead of general cavitation.

General cavitation may be defined as the result of the general lowering of pressure in the turbine until the pressure at some point, or over a certain area of each turbine blade, is equal to the vapor pressure. With such reduction, general cavitation is unavoidable regardless of how well designed a turbine may be. However, with cavitation test data at hand, it is a simple matter for the power-house designer to so locate his turbine with respect to tailwater that the possibility of general cavitation will be eliminated. On the other hand, local cavitation is the fault of poor design, and is likely to cause serious pitting even when the general pressures in the turbine are well above the danger limit. There are numerous causes for local cavitation, such as too abrupt curvature in the blade surfaces, reversals in curvature in the direction of flow, roughness or small obstructions, and excessive runner clearances. Local cavitation may also be caused by disturbances of the turbine from upstream such as might be set up by poorly designed wicket gates, gates that overhang the throat ring at or near full opening, poorly shaped stay vanes, stay vanes at improper angles with the flow, and poor design of curb ring, head cover, and scroll case. General cavitation of a turbine is affected by blade area, hub diameter and shape, and draft-tube design. It is evident that the more efficient the draft tube, the lower the general pressure on the turbine will become for a fixed set of head and discharge conditions.

Present knowledge of hydraulic theory and mathematical analysis has proved a valuable aid, but it has its limitations, and the refinements of design must be attained by a "cut-and-try" process in the laboratory. The location of the causes of local cavitation in the cavitation laboratory is extremely difficult and costly since there are relatively few means for detecting its presence. Some work has been done with stroboscope, photography, spark photography, and stereoscopic photography; but all of these methods require clear visibility of the parts affected.

Much valuable information on local cavitation has been obtained by means of paint tests. For such a test the runner is painted and then the test is made before the paint has completely hardened. Where cavitation occurs, the paint is removed. This has been confirmed by experiments where actual pitting on the prototype has been simulated by means of paint tests. This is a rather delicate test to make, since, if the paint is too hard, it will require a long period of running to show the effects of cavitation, and if it is too soft, the high-water velocities may remove the paint by erosion.

The Corps of Engineers, U. S. Army, used models made of a transparent material for making intake, scroll case, and wheel-setting tests for the Bonneville Development. Sawdust with the same specific gravity as water was introduced into the flow, and from slow-motion movies it was possible to trace out the flow lines and detect the presence of eddies. No doubt this method of testing would lead to a quick solution of many of the causes of local cavitation. However, such tests must be made in conjunction with efficiency and cavitation tests. In some cases guiding vanes may seem necessary to remove a disturbance from the flow, but efficiency tests are likely to show that the surface friction losses of the guides are far greater than the gain from eliminating the eddy. Experience seems to indicate that the most efficient intake and scroll case is the one with the fewest piers and obstructions.

As is often the case in a new field of research, certain unjustified liberties were taken in some of the early model tests. At one time it was customary to conduct tests on a model turbine in an open-flume setting with a straight conical draft tube and then attempt to predict the behavior of the prototype having an intake, scroll case, and elbow draft tube. No doubt this practice is responsible for many of the disappointing efficiency results that have been obtained by field tests. Present practice for important installations requires model tests on precise models of the entire water passages from the intake to the draft tube except where the distance from intake to scroll case is great.

EFFICIENCY AND POWER STEP-UP

After the efficiency characteristics of a turbine model have been determined by model tests, it is desirable to be able to predict the characteristics of the prototype. Ordinarily, the percentage of losses in a turbine varies inversely with the size. Thus, in going from model power and efficiency to prototype power and efficiency, an increase in excess of that computed from the theoretical step-up, due to size, can be expected.

There are a number of variable factors which affect the step-up, such as surface roughness of the model as compared to prototype, and the mechanical losses of the laboratory set-up compared to the prototype. For this reason it would seem logical that one step-up relation would not hold exactly true for all laboratories or for all models in the same laboratory. Most models at present are carefully made of bronze and considerable care is taken to give them exceptionally smooth surfaces; so, as a rule, most models have comparable surface roughness.

Since the Holtwood laboratory has been in operation, it has been possible to make direct comparisons with efficiency data taken in two other laboratories. In one case the agreement was within 0.2% over the entire range and in the other case the peak efficiencies agreed, but there was a slight shift in the curves. The usual test head for the Holtwood laboratory is from 35 ft to 50 ft, whereas the other two laboratories tested at 3-ft to 4-ft head (I. P. Morris) and at 8-ft to 10-ft head (S. Morgan Smith Co.), respectively. Because of the wide range of testing heads, the three laboratories must necessarily have radically different test equipment. Yet, in spite of this, one step-up formula should give good results for the three laboratories, for all practical purposes, since the head factor is usually negligible.

The most widely known step-up formula[13] in the United States is one developed about 1925 by Prof. L. F. Moody. This formula is based on a theoretical analysis of the reduction in losses with increase in size. The Moody step-up formula is:

$$e_p = 100 - (100 - e_m)\left(\frac{D_m}{D_p}\right)^{0.25}\left(\frac{H_m}{H_p}\right)^{0.01} \quad\ldots\ldots\ldots\ldots(9)$$

in which: e_p = expected efficiency of prototype; e_m = model efficiency; D_p = diameter of prototype turbine, in inches; D_m = diameter of model turbine, in inches; H_p = prototype head, in feet; and H_m = model head, in feet. Equation (9) applies only to the peak efficiency of the model even if it occurs at a value of ϕ outside of the operating range. To obtain the expected efficiency at off-peak points, the increment of efficiency obtained at the peak efficiency point is added to the model efficiency at those other points.

When applied to complete homologous models, the Moody formula gives results which agree well with field tests for small and medium diameters. From various experience curves that have been developed, it appears that, for large-size turbines, the step-up is somewhat less than that obtained by the Moody formula. Few of the large propeller-type turbines that have been installed in the United States recently have been tested in the field. This is extremely unfortunate since failure to know what actual field efficiencies have been attained may delay advances in design.

In an analysis of step-up of turbine efficiency due to size, W. S. Pardoe, M. Am. Soc. C. E., found that the exponent applied to the ratio of diameters, theoretically, should be 0.20 instead of 0.25. If the Moody formula is modified according to Professor Pardoe's analysis, much better agreement is obtained between the stepped-up efficiency and the field efficiency for the larger-diameter installations that have been tested in the field. From the data on hand it appears that for the smaller diameters (say, less than 150 in., for which most of the comparative data exist) the exponent of 0.25 gives closer agreement with the field tests than the exponent of 0.20. This tendency might be explained by the fact that all results where the field test reached or exceeded the expected efficiency were used in the experience curve, whereas results that fell short of the expected efficiency were discounted and not plotted.

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[13] "The Propeller Type Turbine," by Lewis F. Moody, *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 625.

The Moody formula gives only the efficiency step-up without any attempt to determine what step-up there may be in power. In discussing the subject of power step-up in 1936, F. H. Rogers,[13] assistant chief engineer, Baldwin Southwark Corporation, made a very logical suggestion. It was his opinion that the increase in efficiency of the prototype is the result of a decrease in the internal loss in head, due primarily to the larger water passages. On this basis the increase in efficiency of the prototype is due to a greater effective head, causing an increase in the quantity of water but a larger increase in the power output.

To determine the relative increase in power output as compared to water quantity, let h = head on model, Δh = gain in head on prototype, e_m, Q_m, P_m = efficiency, water quantity, and power output for model, and e_p, Q_p, P_p = efficiency, water quantity, and power output for prototype. Then:

$$e_p = e_m \frac{h + \Delta h}{h} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (10a)$$

$$Q_p = Q_m \frac{h + \Delta h}{h}^{0.50} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (10b)$$

and

$$P_p = P_m \frac{h + \Delta h}{h}^{1.50} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (10c)$$

For the usual small values of Δh this means that the percentage increase in power output will be about three times the percentage increase in the quantity of water and about one and one half times the increase in efficiency.

An analysis of a large number of comparative model and prototype tests on Francis turbines showed that, although the efficiency step-up agreed reasonably well with the Moody formula (Equation (9)) when the turbines were truly homologous as far as could be ascertained, the power varied from a negative 6% to a positive 10% from the value obtained by multiplying the model power by the ratio of heads to the three-halves power and the ratio of diameters squared.

From the rather meager comparative data available on propeller-type turbines, there seems to be a definite step-up in power which agrees with the value predicted by Mr. Rogers.

Turbine designers agree that in all probability failure to get a set-up in power is due to the fact that the prototype is not exactly homologous to the model. The form of the propeller turbine is much simpler than the Francis turbine, and it is, therefore, much easier to make the model and prototype homologous.

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[13] *Transactions*, A. S. M. E., May, 1936, p. 327.

It appears that, for propeller-type turbines, the expected power can be predicted accurately, but for Francis turbines extreme caution must be exercised to insure that the model and prototype are homologous before an accurate prediction can be expected.

FIELD TESTS

As the demand for hydroelectric power increased, it became increasingly more important to use the available water at a given development in the most efficient manner. In certain developments, also, governmental requirements called for an accurate accounting of the water used. Consequently, it became vitally important to determine the actual efficiencies of the units. The electrical output could be measured well within 1% and, by suitable tests, the mechanical and electrical losses of the generator, turbine, and bearings could be determined accurately, giving a value of turbine output within 1% of the true value. However, until comparatively recent times, the difficulties of measuring large volumes of water within 1% seemed almost insurmountable.

Early tests with pitot tubes and the salt-titration method gave reasonable results, but both of these methods become unwieldy when the size of the conduit passes medium size and for medium or large volumes of water. The salt-velocity method[14] and the pressure-time method[15] of measuring the flow of water were introduced in 1923. Both of these methods were checked carefully against laboratory standards and were found to give accurate results. These methods are widely used in North and South America and, for suitable conditions, they give results closer than 1%.

Since it depends on the phenomenon of water hammer, the pressure-time method can be applied only to the flow of water in closed conduits. The salt-velocity method can be used in both open channels and closed conduits. Both methods used a section of conduit as a test section, and it is essential that the cross-sectional areas and lengths of the section be known accurately. It is also essential that there be no "dead water" in the test section. A number of important low-head plants have been installed with extremely short intake passages. In some of these developments it is impossible to get sufficient length of water passage to obtain accurate results by either method.

When the Safe Harbor Development was started, it was considered that none of the accepted methods of water measurement in use in the United States would give accurate results; consequently, a survey of methods in use in Europe was made. It was found that a multiple-current-meter method, using meters of the axial flow type,[16] was used almost entirely.

The general procedure followed is to install a frame which travels in the gate slots of the intake. A sufficient number of meters are mounted along a horizontal streamlined member of this frame to insure an accurate horizontal profile of the velocity. By taking sufficient readings of velocity at various positions vertically, the average velocity of the entire passage is determined. One horizontal velocity profile is taken by simultaneous readings of the meters

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[14] "The Salt Velocity Method of Water Measurement," by C. M. Allen and E. A. Taylor, Members Am. Soc. C. E., presented at the Annual Meeting of the A. S. M. E., December 3–6, 1923.

[15] "The Gibson Method and Apparatus for Measuring the Flow of Water in Closed Conduits," by N. R. Gibson, M. Am. Soc. C. E., Paper No. 1903, *Transactions*, A. S. M. E., Vol. 45 (1923), p. 343.

[16] *Transactions*, Am. Soc. C. E., Vol. 95 (1931), Fig. 4, p. 773.

This method of water measurement is believed to result in too small a value of discharge, for reasons that have been considered carefully. Ratings in still water with the meter turned at an angle to the direction of travel show invariably that the spoke-type axial-flow meters, such as those recommended for turbine tests, register less than the component of the velocity in the direction of the meter axis, which is obviously the velocity of travel in the water, times the cosine of the angle at which the meter is turned. Not only is this a demonstrated fact, but a graphical analysis of the angle of attack of the water on the blade, when in two opposite positions of its rotation, show that, for reasonable angles of inclination, the degree of under-registration agrees with that which would be expected.

Few intakes of modern units have walls that are parallel at the measuring section, from which fact there is necessarily some obliquity of flow. Consequently, the meter is expected to register, not the true velocity, but the component perpendicular to the measuring section, which from the still-water ratings, and from theoretical analysis, cannot be done accurately by any known meter. Some attempts have been made abroad to turn the meter to agree with the theoretical streamline, and then to correct for the angle that this makes with the metering section.[17]

The situation becomes even more complicated when it is considered that turbulence may cause the meter to be buffeted by a multiplicity of impulses coming possibly from widely varying angles and directions; and no matter how much falsework may be constructed to straighten the flow, there are always eddies and cross-currents.

Recognizing the fact that some meters under-register the flow, a valuable suggestion was made by W. Monroe White, M. Am. Soc. C. E., that a meter known to under-register might be used in conjunction with one that is known to over-register; and, by combining the results, the true flow might be determined. The suggestion proved impractical for the simple reason that there is no meter which, when rigidly supported, will over-register consistently the same amount for a given angularity regardless of the direction from which the flow comes. Nevertheless it was the basis for an important technical advance in the measurement of water.

Failing to find two meters that had opposite characteristics, the next step which followed very logically was to find two under-registering meters that had differing characteristics. J. M. Mousson demonstrated[18] that the apparent failure of certain current meters to agree with the volumetric standard could be accounted for easily by their differing oblique flow characteristics in spite of the fact that the channel upstream from the meter had a uniform cross section for a long distance. Later he demonstrated[19] that the quantity of under-registration of spoke-vane meters of the same family was directly related to the pitch, and again graphical analysis of the angle of attack explained within reasonable limits the results that were actually observed in the rating tank.

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[17] "Eine neue Anwendung des Flugelmessverfahrens bei den Abnahmeversuchen im Limmat-Kraftwerk Wettingen," by C. F. Streiff and H. Gerber, Schweiz-Bauzeitung, Vol. 103, No. 3, pp. 36–39.

[18] Discussion of paper "Research Institute for Hydraulic Engineering and Water Power," by Hunter Rouse, Assoc. M. Am. Soc. C. E., *Transactions*, A. S. M. E., Vol. 55, No. 10, August, 1933, pp. 35–39.

[19] *Transactions*, A. S. M. E., February, 1936, Vol. 58, p. 141.

The outstanding fact that was revealed, both experimentally and on paper, was that, regardless of the angle at which a spoke-vane meter is inclined to the actual movement of the water relative to it, the extent of under-registration of one meter in comparison with that of the other is in direct proportion to the pitch.[19] Consequently, assuming that both meters are exposed to exactly the same conditions of flow, the difference in registration becomes a measure of the extent to which either of them under-registers the true flow. It is interesting to note (again within reasonable limits) that the angle at which the water strikes the meter, whether uniform or constantly varying in amount and direction, drops completely "out of the picture" since by the law of averages both meters will be subjected to the same average conditions in the course of a test.

To be sure, this method falls short of Mr. White's ideal for the reason that, unfortunately, it is necessary to extrapolate outside of two readings rather than to interpolate between them; but at least it makes use of means at hand instead of abandoning the effort because something is needed that does not exist. This method was first used[20] in making turbine efficiency tests on the 42,500-hp Kaplan turbines at Safe Harbor.

Since this method of water measurement was new, it was deemed advisable to compare its accuracy with some better-established method. After numerous experiments at Niagara Falls, N. Y., N. R. Gibson, M. Am. Soc. C. E., found that he would have confidence in his ability to measure, satisfactorily, the discharge of the Safe Harbor units with the length of test section available. Consequently, two units were equipped with the necessary piezometer taps, and the tests were conducted. The agreement between the Gibson tests and the two-type, current-meter tests was excellent, being less than 1%. The maximum discharge of these units is about 10,000 cu ft per sec.

In spite of the excellent agreement between these two comparative tests, there is still doubt in the minds of some hydraulic engineers regarding the fundamental principle of the two-type, current-meter method. These doubts arise partly from the assumption that turbulence in flowing water affects the meter the same as angularity in the still-water rating. To date (1939) there has been neither conclusive proof that this assumption is correct, nor proof to the contrary.

An acceptance test by the current-meter method requires rather expensive equipment, most of which cannot be adapted, economically, to tests on other developments. For this reason, it may be anticipated that the cost of a current-meter test will be considerably greater than a test by the Gibson or Allen method. Consequently, the use of current meters will be limited to low-head developments with extremely short intake passages where the other two methods cannot be applied.

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[20] "Water Gauging for Low Head Units of High Capacity," by J. M. Mousson, *Transactions*, A. S. M. E., August, 1933.

Messrs. Allen and Gibson have found from experience that years of study and practice are necessary to train men to conduct tests successfully by their methods. The two-type, current-meter method is no less intricate than the other two methods and requires every bit as much training before an engineer can be considered competent to conduct tests. To a large extent the success of the Allen and Gibson methods has depended upon the close control which they have kept over the application of their methods, permitting only competent assistants to conduct tests that they were unable to supervise personally. Since the two-type, current-meter method is not a patented process, no such strict control is possible, although highly desirable.

Conclusions

The engineering profession is quite likely to have a self-satisfied feeling regarding the state of the art of turbine design similar to the feeling which was prevalent about 1914 when turbine efficiencies of more than 90% were attained. However, a review of the progress during the intervening years will show the fallacy of that feeling, and a study of the possibilities for improvement on present designs should spur engineers to renewed efforts.

As in the past, the laboratory is the logical place to study means of improvement, although theoretical studies should not be neglected. Since the margin of possible gain is relatively small, it is of considerable importance to take every precaution to eliminate all sources of error in the laboratory. In the past there has been the tendency to make assumptions in order to reduce the cost of tests, finding later that the accuracy of the tests was questionable because of the unjustified assumptions. It is equally important to make tests on the prototype so that there will be assurance that the expected performances are actually attained.

It would be of great assistance to the profession if each manufacturer would make a study of step-up with an attempt to determine whether the exponent for the ratio of turbine diameters in the step-up formula should be 0.25, as used by Moody, or 0.20 as indicated from the theoretical analysis by Professor Pardoe.

DISCUSSION

W. S. Pardoe,[21] M. Am. Soc. C. E. (by letter).—This Symposium is a concise and readable summary of turbine testing methods brought up-to-date. The only reason for entering this discussion is in connection with Professor Moody's step-up formula, mentioned by Mr. Davis.[12] The only difference between that formula and the one suggested by the writer is the difference between a fourth and a fifth root. The writer will state his reasons for suggesting the latter, or fifth, root.

Rankin's expression for the loss of head in pipes derived by dimensional analysis is

$$\Delta h = k \frac{L}{d^{3-n}} V^n \text{.......................} (11)$$

The value of n for smooth brass pipes is usually given as 1.75 (some experiments by the writer, on 1.5-in. brass pipe, gave $n = 1.79$). The value of n for rough pipes approaches 2.0; and n, therefore, lies between 1.75 and 2.

Equation (11) may be rewritten in the familiar Darcy form as

$$\Delta h = K \left(\frac{w}{V \rho d} \right)^{2-n} \frac{L V^2}{d \, 2 g} = f \frac{L V^2}{d \, 2 g} \text{.............} (12)$$

If d is the only variable and $n = 1.75$, $\Delta h \propto \frac{1}{d^{0.25}}$ and Professor Moody's step-up formula follows; but who will contend that the inside of the prototype is of the same physical smoothness as the model? This is necessary, as Blasius' expression

$$f = \frac{0.3164}{\left(\frac{V \rho d}{w} \right)^{0.25}} \text{.......................} (13)$$

was obtained for smooth pipes. Furthermore although the writer has the highest respect for the way in which the turbine manufacturers get the water through with very small losses, nevertheless there will be some small losses due to impact and sudden enlargements which will vary as V^2 and hence will not vary with d.

In plotting coefficients for venturi meters for the Fluid Meters Committee of the American Society of Mechanical Engineers from about one hundred weighed water tests of venturi meters varying from 1 in. to 16 in., the writer expressed the lost head between the main and throat as

$$\Delta h_2 = k \frac{V_2^2}{2 g} \text{.......................} (14)$$

and found the empirical expression,

$$k = \frac{0.045}{d_2^{0.23}} \text{.......................} (15)$$

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[21] Prof., Hydr. Eng., Civ. Eng. Dept., Univ. of Pennsylvania, Philadelphia, Pa.

With the foregoing considerations in mind, the writer suggested the fifth root as more conservative than the fourth, which he thinks is too liberal to the manufacturers. An example will show the difference, using Fig. 16. The maximum efficiency $e_m = 83.6\%$. Then, by Equation (9):

$$e_p = 100 - (100 - e_m)\left(\frac{16}{200}\right)^{0.25} = 91.3\%$$

and

$$e_p = 100 - (100 - e_m)\left(\frac{16}{200}\right)^{0.20} = 90.1\%$$

which is a difference of 1.2%, but on the safe side for both customer and manufacturer.

No amount of abstract reasoning can take all the variables into account. The turbine manufacturers should have, or will soon have, sufficient reliable data on which to base a satisfactory step-up formula. The step-up formulas for discharge and horsepower suggested by Mr. Rogers (Equations (10)) are subject to certain minor corrections and could be derived a little more rationally, thus, if H is total head and Δh is the lost head: $e = \frac{H - \Delta h}{H}$; $\Delta h = H(1 - e)$; and the useful head $= H - \Delta h = eH$. Therefore,

$$\frac{Q_p}{Q_m} = \left(\frac{e_p\, H_p}{e_m\, H_m}\right)^{0.5}\left(\frac{D_p}{D_m}\right)^2 \ldots\ldots\ldots\ldots\ldots\ldots (16)$$

and

$$\frac{P_p}{P_m} = \left(\frac{e_p\, H_p}{e_m\, H_m}\right)^{1.5}\left(\frac{D_p}{D_m}\right)^2 \ldots\ldots\ldots\ldots\ldots\ldots (17)$$

Donald H. Mattern,[22] Assoc. M. Am. Soc. C. E. (by letter).—Symposiums, such as this one presented so capably by Messrs. Winter and Davis, should open up discussion upon present-day hydraulic turbine practice. Interest in refinements of design and in the checking of the different assumptions by test has been increasing, due not only to the economics of the situation but to the trend of technological thought toward research.

Design.—Mr. Winter has made some interesting comments upon the different phases of a hydroelectric development. Throughout the entire paper he emphasizes concentration upon refinements of design as a further means of reducing hydraulic losses in the turbine water passages. It should be pointed out, however, that often the cost of securing desired flow conditions may amount to more than the capitalized value of the benefits received. In a drive toward technical perfection, it might be quite easy to lose sight of the economics of the situation, and each case should be studied by itself and decisions made accordingly.

Examination of plans for the Bonneville intake, scroll case, and draft tube, as reproduced in engineering literature, gives one the impression of complexity and an implication of difficult construction. It would be interesting to learn the actual cost of intake, scroll case, and draft tube as compared with a more simple design. Differences in $h_f + h_v$ for the unit as built, and for the conventional setup as originally tested, would be of assistance to those who may have similar problems. That there appears to be some difference of opinion between designers as to the value of additional structures in the intake and scroll case is evidenced by Mr. Davis' comment that "Experience seems to indicate that the most efficient intake and scroll case is the one with the fewest piers and obstructions" (see second paragraph preceding heading "Efficiency and Power Step-Up").

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[22] Assoc. Civ. Engr., Tennessee Valley Authority, Knoxville, Tenn.

It has not been proved that splitters improve the efficiency and performance of a draft tube. Some well-qualified turbine experts say that splitters make the unit perform more smoothly and also improve the efficiency. Other equally qualified engineers claim that splitters do not help the operation of their turbines. Both groups have test curves to prove their individual contentions. To the average engineer, therefore, it would appear that, until definite information is available, he should choose the draft tube recommended by the specific turbine manufacturer, but should look with question upon any design that will increase his costs.

Cavitation.—As stated by Mr. Davis, development of high-speed turbines has resulted in lower pressures within the water passages and has placed more emphasis upon a critical cavitation coefficient. Pitting can be decreased greatly if the runner is set low enough to secure a sigma value greater than that which is critical. The necessary extra excavation is not desirable, however, because of the high cost per unit length of draft tube required by the large discharges. Just how high the runner can be placed above tailwater, therefore, depends chiefly upon experiments made on model runners. By the use of reliable test data it is possible, safely, to raise the runner higher than would otherwise be permitted by the manufacturer. In other words, factors of safety can be anticipated more closely and the resultant savings passed on to the customer.

With regard to cavitation laboratories, Mr. Davis has emphasized the fact that, originally, it appeared necessary to test the model under the same head conditions as exist in the field, and that later developments have shown this to be unnecessary for heads greater than 10 m. In tests conducted at Holtwood in 1938, the question arose, and different sigma curves were run at varying heads in order to check this point. These test heads varied as much as 10 ft at the same sigma point, but showed no difference in regard to the location of critical sigma. Still later tests, which were computed at the new Baldwin-Southwark cavitation laboratory under approximately 20-ft to 30-ft head, checked very closely with an homologous runner that had been tested at Holtwood under a much higher head. From these very meager data, it would appear that for practical test purposes Prof. E. W. Spannhake's upper limit could be lowered to approximately 7 m. Further data are desirable, and any assistance given by suitably equipped laboratories would be appreciated by the profession.

The curves shown in Fig. 16(*a*) illustrate possible cavitation test results. In them the efficiency, horsepower, and discharge breaks lie in a vertical line.

The writer's experience indicates, however, that they are more ideal than typical, although it is true that breaks like those illustrated do occur at times. Performance at the point of critical sigma does not necessarily result in an increase in discharge, a decrease in power, and a resultant drop in efficiency. In tests conducted at two different laboratories, on two different units, it has been noted that there are cavitation curves where the discharge has decreased almost directly with the power. The result is that efficiency has dropped little, if any, at the lower sigma values. On other curves, the discharge remained practically a straight line with all the changes in efficiency characteristic being caused by variations in power. The question might be asked whether the discharge characteristic is not more dependent upon laboratory idiosyncrasies than upon other factors.

Breaks in the sigma-horsepower curves have been found to be much more consistent. Using these horsepower breaks, the doubt connected with a proper power step-up to the large unit is eliminated. Since cavitation is tied up with the difference of pressures between the two faces of each blade, limitations of unit generation based upon horsepower, stepped up from the corresponding break in the model curve according to the three-halves power of the head, appear to be more conservative than using the discharge break.

Modification of limits so determined should be made as a result of field observations extending over a period of time long enough to cover an operating head cycle. Because of the nature of the step-up from model to prototype, these field tests will normally lower the point of critical sigma.

Efficiency and Power Step-Up.—Mr. Davis states that prediction of prototype performance from model tests is of particular interest in connection with many of the low-head plants now being constructed. There is some doubt whether a good test can be obtained by either the Allen or the Gibson methods because distances from their intakes to the center line of units are so short. In many instances, the effective metering sections are less than 40 ft, or less than that required for an accurate water measurement.

Many hydraulic engineers believe that accurate results cannot be secured by current-meter tests without high costs. Mr. Davis' treatment of the methods used at Safe Harbor is interesting, but similar applications must be made successfully elsewhere before complete acceptance will be given. As a result, there is no satisfactory testing method available for use at these low-head projects.

Several acceptance tests of propeller-type turbines have been run upon completely homologous models, and acceptance or rejection of the prototype determined upon this basis. Probably the first such test conducted was made in July, 1935, upon a fixed-blade propeller turbine having a diameter of 16 in. and a scale ratio to the prototype of 1 : 16.5. The model efficiencies were stepped up by application of the Moody formula (Equation (9)) and compared with the guaranteed values. Final power guarantees, however, were based upon index tests conducted in the field.

With regard to step-up of efficiencies, it should be noted that the head factor given in the formula is not being used at the present time by Professor Moody or by several of the turbine manufacturers. A test witnessed by the writer is probably one of many which tends to show that this part of the equation can be eliminated. The model was first tested under heads from 8 to 10 ft and then under the 35 to 50 ft which exists at Holtwood. Under unit speed conditions, corresponding to normal operating heads, the efficiencies between the two laboratories checked within close limits.

Professor Moody's original presentation of a proposed step-up formula[12] did not specifically state how the method should be applied to test data. Different laboratories have been using the formula in different ways, and, although the peak efficiency for the best speed is the same, there is a variation at other parts of the performance range. An example showing effects of three methods of applying the formula to a Kaplan unit is shown in Fig. 18. For illustrative purposes, efficiencies are shown as percentages of the peak, and the power as arbitrary values. Curve *A* demonstrates the method recommended by Professor Moody, and was obtained by computing the efficiency increment for the turbine, irrespective of speed or blade angle, and adding the constant figure to the envelope efficiency for any head curve covered in the test. Curve *B* was secured by applying the Moody formula to each point of the model envelope. Curve *C* was obtained by computing the peak prototype efficiency for each blade angle, and then finding the ratio of that value to the peak model efficiency. All points of that blade angle were multiplied by this factor to secure the prototype efficiency. The envelope curve for the unit was then drawn through these stepped-up values. Although there is not much difference at the head corresponding to the best turbine speed, the variation becomes more apparent at other heads. Curve *B* is always highest and flattest, curve *A* the lowest, and curve *C* is usually between the others.

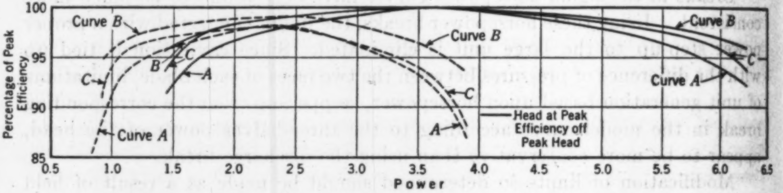

FIG. 18.—THREE WAYS OF APPLYING EQUATION (9) TO A KAPLAN UNIT

It would be of value to the profession if the turbine manufacturers would cooperate not only in arriving at proper exponents to be applied to the ratio of runner diameters, as suggested by the author, but also in adopting the most nearly correct method with which to apply the formula to off-peak efficiencies. The manufacturers are the only ones who have information concerning the large number of installations which are needed to form the basis for such formula alterations.

Mr. Winter suggested another method of estimating the field performance when an accurate model test has been made, but failed to give any details of what he calls the "power step-up" method. Discussion of this method, together with several examples, would comprise a valuable contribution to the symposium.

Runaway Speed Tests.—Testing of model units for runaway speeds under different possible operating conditions is of considerable value. In 1935, J. D. Scoville showed[23] that runaway speed varies with sigma as well as with the usual factors of head and diameter of runner. To have a true picture of the overspeed conditions for which the generator should be designed, the speeds must be compared for the different sigmas that can occur under critical head conditions. Such model tests now form a part of several acceptance test programs, and, when conducted ahead of the purchase of the generators, may make it possible to buy the electrical equipment more economically.

Field Tests.—No discussion of field tests is complete without comment upon index methods of checking turbine performance. For practical purposes, it should be noted that a gate opening-output curve is sufficient to determine the best point at which to operate an individual unit. For more than one unit in a plant, and for Kaplan units, the relative efficiencies may be secured by including scroll-case differential pressure data with the output information.

Taps to indicate differential pressures may be placed either in the turbine speed ring columns, or in the scroll case itself. The latter type, in which one tap is placed in the outer wall of the scroll case and the other one in the casing just above the speed ring, is illustrated in Fig. 13, and is thought by many engineers to give more consistent results.

An attempt was made to coordinate model and field index tests at one plant by putting taps in the model at the same position as they were installed in the large unit. Care was exercised in locating the piezometer taps and in reading the water columns during the model tests, but discharges did not step up as would be expected for the prototype. Further research into correlation between model and prototype discharges might prove of value.

LEWIS F. MOODY,[24] ESQ. (by letter).—A wide range of subjects is treated in the interesting paper by Mr. Winter. The writer will confine his discussion to only one of them—centrifugal pumps operating as turbines.

In 1931 and 1933, models were tested in the I. P. Morris laboratory embodying certain ideas of the writer (see U. S. Patent 1,919,376 for a detailed description of the principles applied) for the construction of a reversible pump-turbine unit suitable for pumped storage plants. The runners assumed forms intermediate in proportions between pump and turbine practice. As Mr. Winter has found in the tests he mentions, surprizingly high turbine efficiencies were obtained; and the teachings of these experiments has had a material effect on subsequent practice in turbine design. The models tested were complete turbines with wicket gates and draft tubes. Fig. 19 shows the results obtained in the laboratory on these models.

In comparing the results of the reversed operation of pumps, cited by Mr. Winter, with turbines of normal design, several important factors should be considered. First, the efficiency as a turbine needs to be discounted somewhat due to the absence of a draft tube. The efficiencies based on net head on the machine itself, exclusive of velocity head in the discharge pipe, will be reduced when charging the unit with the duty of regaining the velocity head in the pipe, the function performed by the draft tube. This correction under the conditions of most of these tests at the Pasadena laboratory will be of the order of a little more than 0.5%, which is not great but has some effect on the comparison.

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[23] "Cavitation Testing of Model Hydraulic Turbines and Its Bearing on Design and Operation," by L. M. Davis; discussion by J. D. Scoville, *Transactions*, A. S. M. E., Vol. 58, 1936, p. 323.

[24] Prof. of Hydr. Eng., Princeton Univ., Princeton, N. J.

A more important factor is a consequence of the abnormally small eye or throat diameter of pump impellers of the best design. For turbine specific speeds within the range of about 25 to 30, pump impellers which will give these specific speeds will have an eye diameter of about seven tenths to eight tenths of the throat diameter of normal turbines of the same specific speeds. When it is taken into account that the velocity head at the throat varies inversely as the fourth power of the diameter, it is found that the reversed pump will have from two and one half to four times the throat velocity head of the turbine of normal design. Naturally, this greatly increases the cavitation factor of the reversed pump, as compared to a normal turbine, and limits it to installation requirements which could be met by normal turbines approaching perhaps double its specific speed.

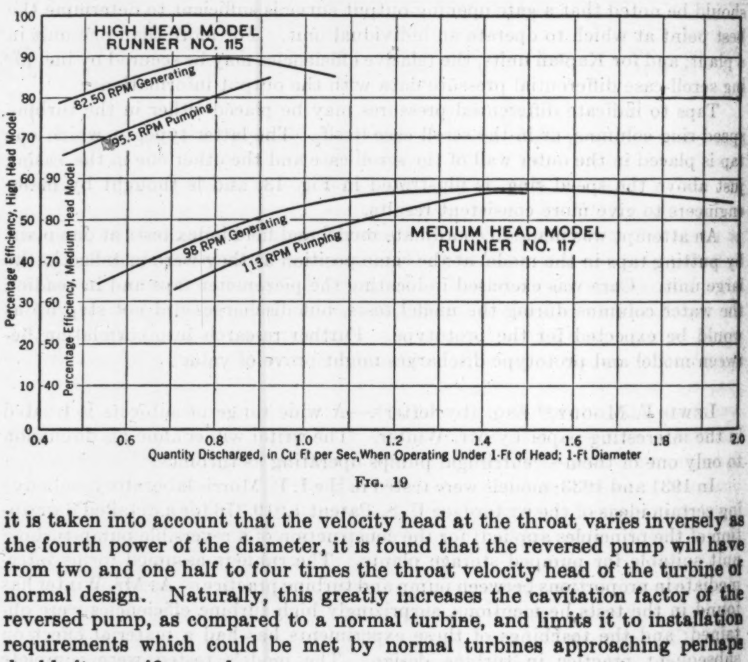

FIG. 19

Still another factor is the abnormal proportions required of the turbine due to the large outside diameter of the pump impeller when compared to a normal turbine. Within the range of specific speeds mentioned, the reversed pump impeller will be about 20% larger in diameter than a turbine of the best normal design. Since the flow in the casing must encounter stay vanes and wicket gates, it is not advisable, in a turbine, to use casing velocities approaching those in pump volutes, or appreciably higher than now normally used. Consequently the over-all sizes of the turbine will be increased materially if the proportions of the pump impeller are adhered to, with a corresponding increase in cost. In a pump impeller operating as a pump, the large outside diameter is necessary to permit sufficiently gradual deceleration of the relative velocities; in a turbine runner there is no such requirement, since the flow is accelerating, and more favorable velocities result from a considerable reduction in diameter.

In comparing the effects of such factors, the only conclusive test would be to compare complete turbine models of normal and abnormal design under equal test conditions in the same laboratory, a procedure which has been the standard method of research in the development of the turbine to its present state.

Passing to the paper by Mr. Davis, the writer finds himself in such general agreement with this valuable paper that he will confine his discussion to a single point—the prediction of field efficiencies from model tests.

When the formula for "stepping up" efficiencies from model to prototype was first introduced, it was expressed in the form

$$\frac{1-e}{1-e_1} = \left(\frac{D_1}{D}\right)^p \left(\frac{H_1}{H}\right)^q \text{.................}(18)$$

with the exponents p and q to be determined empirically from all available test comparisons. The data then available seemed to indicate an exponent p very close to 0.25 for the diameter ratio, and a value for q of the order of 0.01. The effects of head, however, could be determined only roughly since no tests were available to show, directly, the effect of head variation on a given turbine. The expression was then used in this form (as given in Equation (9)) for several years. The writer felt, however, that there was not sufficient information available to support the exponent q used for the head ratio; and on theoretic grounds he decided to drop the head term completely—that is, to use the exponent $q = 0$; and when the formula was published in a paper presented before the Society in 1925,[12] the term was omitted intentionally, giving

$$\frac{1-e}{1-e_1} = \left(\frac{D_1}{D}\right)^{0.25} \text{.....................}(19)$$

The reasons for omitting this term were the considerations that if turbine water passages can be classed properly as "rough conduits," under the designation given by L. Prandtl and Theodor von Kármán,[25] M. Am. Soc. C. E., then, at the high Reynolds numbers representing the turbine water passages, even in the models, viscous forces should be negligible and the losses should vary directly as the square of the velocity. It will be found, for example, that even in a 16-in. propeller turbine model, under a 3.5-ft head, the Reynolds number is of the order of 500,000.

Even with a very large variation of head it has been impossible to detect any effect of head. Thus, a model of the 35,000 hp Oak Grove turbine gave 87.6% efficiency under a head of less than 4 ft. When stepping up from the 12.065-in. diameter to the 50-in. diameter of the large unit, Equation (19) calls for 91.3% efficiency. The actual efficiency in the field, under an 850-ft head, was 91.5%.

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[25] "The Dynamics of Turbulent Flow," by Boris A. Bakhmeteff, M. Am. Soc. C. E., Princeton University Press.

Of course, there is nothing sacred in the numerical value of the exponent 0.25—this was intended to be determined from actual test results. The results secured in the I. P. Morris 16-in. models have been found to step up to field results by the use of this formula with as close agreement as could be expected; and satisfactory agreement has been reported from results in certain other laboratories. It is possible that the efficiencies given by the I. P. Morris 16-in. model tests may be somewhat low, as compared to some other laboratories, due to the effect of bearing friction on the low torque corresponding to the low head.

It should also be remembered that in these model tests the head is measured to tailwater without velocity-head correction, whereas the field tests used to determine the exponent were corrected for the velocity head at the draft tube discharge section. The A. S. M. E. testing code now (1940) in use corrects only for the velocity head in the tailrace. This factor has a material effect on propeller turbines, and tends to reduce the field efficiencies to an extent of approximately 1%. This consideration calls for a readjustment in the empirical exponent in the formula, based on a review of all data now available. Until such a study can be made, an exponent approaching one fifth, recommended by Mr. Davis, seems reasonable from present indications.

It should be realized that, in considering the question of the form of an efficiency step-up formula, conclusions can be reached only from a large mass of comparative test data. Single tests, naturally, will show material variations, since water measurement in large turbine tests is a difficult matter; other factors such as generator output are not always beyond question; exact similarity of model and prototype is not always secured; and uniform accuracy within even 1% error cannot be assumed. In the field of very large units few tests (not more than one or two) are available; and these are not sufficient, in the writer's opinion, to provide a basis for any general conclusions. It scarcely seems reasonable to expect any point of discontinuity, for example, beyond which the formula ceases to apply.

R. E. B. Sharp,[26] Esq. (by letter).—Referring to the heading "Turbines," the writer's experience regarding the shape of the throat ring is at variance with Mr. Winter's statement that "increased power and efficiency * * * may be obtained without the necessity of making the bottom of the throat ring spherical." Fig. 20 shows results of tests as to power, efficiency, and sigma break. The spherical throat ring is indicated by solid lines, and the straight throat ring by broken lines. These tests show that all three of these factors suffer relatively with the straight throat ring except at the highest value of ϕ where the maximum efficiency at that speed shows some improvement with the straight throat ring. However, the maximum efficiency obtained was secured at a lower value of ϕ with the spherical design. In other words, the writer has found that if the spherical throat ring is designed properly, the results secured are superior to those with the straight throat ring. With a properly designed spherical throat ring, the improvement secured with the reduced leakage loss through the clearance space more than offsets the additional burden on the draft tube due to the restricted diameter. Fig. 20 shows the design of a spherical throat ring used in obtaining these results. It is true that improperly designed spherical throat rings may be inferior to the straight throat ring, and where the curvature of the spherical throat ring is incorrect or abrupt there is a decided tendency for pitting to occur just below the restricted diameter.

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[26] Chf. Engr., I. P. Morris Dept., Baldwin Locomotive Works, Philadelphia, Pa.

Centrifugal Pumps as Turbines.—As turbines, these units have a distinct advantage over units designed as such, due to the absence of movable gates surrounding the runner. The reduced friction losses caused by the absence of these gates would tend to account for a high turbine efficiency of centrifugal pumps; and absence of these gates prevents such units from having practicable value as turbines, even for pumped storage projects.

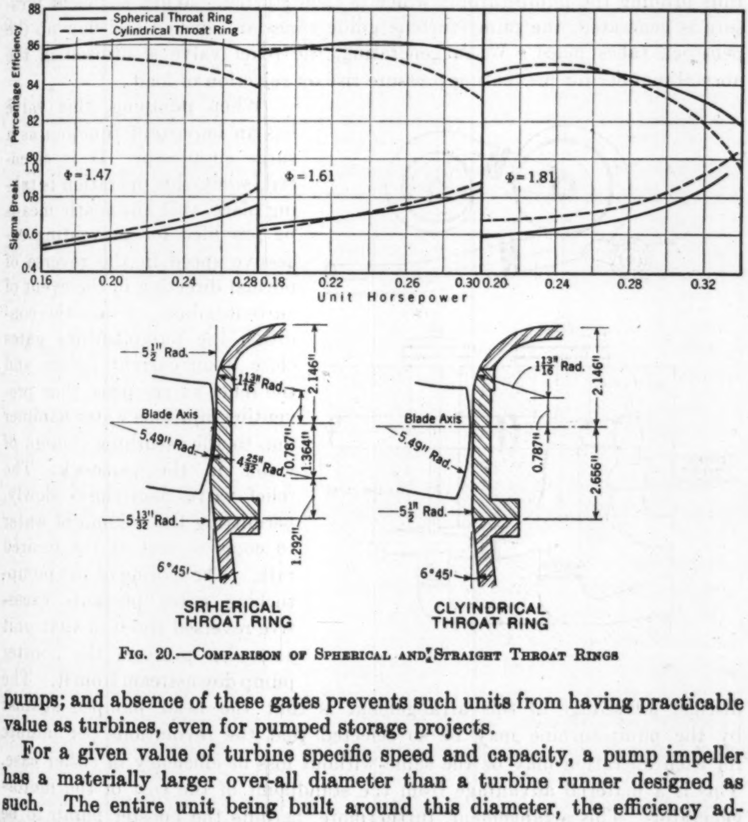

Fig. 20.—Comparison of Spherical and Straight Throat Rings

For a given value of turbine specific speed and capacity, a pump impeller has a materially larger over-all diameter than a turbine runner designed as such. The entire unit being built around this diameter, the efficiency advantage must be great to justify the additional cost resulting from the larger runner diameter.

Studies have been made and models tested in the United States in an attempt to develop a hydraulic unit for use both as a pump and, when run in reverse direction, as a turbine. These units are of the Francis type as the reservoir necessary usually prohibits a low-head pumped storage project. Very good efficiencies (89% when generating and 85% when pumping) have been obtained from these models, when operating both as a pump and as a turbine; however, a two-speed motor-generator is usually necessary where the unit takes the entire head both when pumping and generating.

When the head is greater than about 400 ft, present pump practice requires that more than one stage be used. On this basis, the arrangement shown in Fig. 21 has been devised. When generating, the pump-turbine takes the entire head, valve *A* is open, and valves *B* and *C* are closed. In order to pump, valve *A* is closed, valves *B* and *C* are open, and the booster pump is started, thus priming the pump-turbine which is then started. When sufficient pressure is generated, the pump-turbine guide vanes are opened and flow up the penstock takes place. When generating, the relief valve is utilized in the normal manner for preventing pressure rise on rejection of load.

When pumping, this valve has an important function as a surge suppressor. It is essential, when this operation is taking place, that automatic means be provided for preventing excessive speed in the reverse of normal direction in the event of current failure. Under this condition the pump-turbine gates close upon current failure and the relief valve opens, thus preventing injurious water hammer due to the returning column of water in the penstock. The relief valve then closes slowly, permitting the column of water to come to rest at the desired rate. The closing of the pump-turbine gates prevents excessive reversed speed of that unit and also protects the booster pump downstream from it. The distinct advantage of this arrangement is that the head pumped against by the pump-turbine may be so selected that the revolutions per minute for each operation may be the same without loss of efficiency in either case. This is a material advantage from the standpoint of the cost of the motor-generator. This arrangement, furthermore, permits the booster pump to be placed well below tailwater (as may be desirable to give the desired value of sigma) and permits the pump-turbine to be placed above tailwater, both for the pumping and the generating cycle, with values of sigma which are adequate in each case.

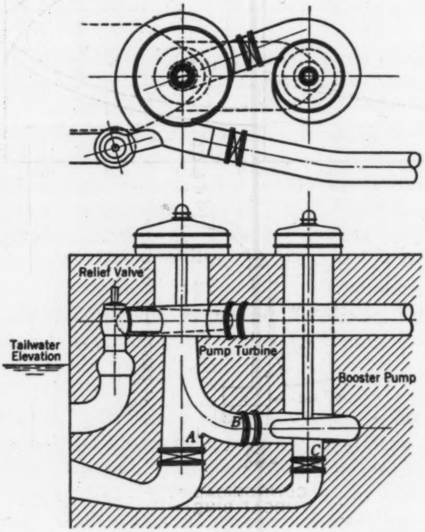

FIG. 21

MARTIN A. MASON,[27] **ASSOC. M. AM. SOC. C. E. (by letter).**—The consideration of trends in hydraulic turbine practice which has been initiated by Messrs. Winter and Davis opens two related fields of discussion which should have an important place in the Symposium.

The importance of model tests in any turbine testing regimen is evident, and such tests admittedly are very useful; but the writer ventures the thought that their usefulness may be rather more limited than recent practice would indicate. Reference is made, of course, to the well-established fact that true similarity of model and prototype is not obtained, requiring, therefore, corrections to be applied to the transfer, or step-up, equations. This fact, in itself, casts considerable doubt upon the value of the model test results when scaled up to the prototype; and when the history of the correction formulas now in use is reviewed there is a tendency to conclude that the present method of predicting prototype results from model tests is open to objection.

It is true that supposedly "theoretical" analyses have resulted in published correction equations to be applied to the efficiency; but the results obtained by the different equations vary among themselves by an amount equal to 10% or more of the step-up they define. Moreover, a critical review of the derivation of these "theoretical" corrections shows that they may not be based on sound assumptions. Specifically, the assumptions regarding similar roughness in model and prototype appear to be questionable, as well as the use of Blasius' formula, which is known to be applicable only to Reynolds numbers of about 100,000 or less.

It would seem more reasonable to base the efficiency correction on experience curves relating model and field tests, rather than, in a sense, on artificial relations which may have little basis in fact. This could be done very simply by turbine manufacturers who, alone, have available the requisite information to define not only the efficiency relations but also the discharge and power relations. For the larger low-head turbines this procedure may not be possible since field-test information for most of the recently installed larger turbines of this type is not available. However, the adoption of a policy of field testing for acceptance in the future, in lieu of acceptance based on model tests, would soon provide sufficient data to allow the formulation of empirical corrections for this class of turbines, or the verification of the formulas used at present. This suggested policy leads to a discussion of field tests of hydraulic turbines.

Much has been written, pro and con, about the various measurement methods cited by Mr. Davis, and voluminous data have been assembled to prove the accuracy of this or that method, with the result that perhaps many engineers have become confused in the tide of claims, counterclaims, information, and misinformation. It might be worth while, therefore, to attempt to recall certain fundamental ideas pertinent to the water-measurement problem.

Each of the numerous methods of measurement that have been developed has some advantage that may or may not be common to other methods; and it is certain that the proponents of each method have satisfied themselves as to its advantages before lending their support to its exploitation. In this respect it is of interest to note in the Test Code of the Société Hydrotechnique de

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[27] Chf., Research Section, Beach Erosion Board, War Dept., Dalecarlia Reservation, Washington, D. C.

France the statement[28] that "even if none of these methods is perfect and universally applicable, there are, nonetheless, very few which are not of value when correctly used."

The writer feels that the poor results obtained in many applications of some of the measurement methods can be attributed to a lack of knowledge, on the part of the operators, of the conditions required for the correct application of the methods; that is, the methods were not "correctly used." A mere similarity of conditions of test is not sufficient to assure correct utilization of a method; it is rather of primary importance that the fundamental principles of the method be understood thoroughly and the proposed application of the method be examined with respect to these principles. In addition, a wide experience with the method seems to be necessary, particularly when the method involved is one of those whose fundamental principles have not been thoroughly delineated. With the knowledge and experience thus defined, one is in a position to hazard a prediction as to the accuracy of a particular measurement under given conditions, whereas without such a background little may be said regarding accuracy.

In the case of the measurement of water in low-head turbine installations, it seems that, with few exceptions, the profession has lost sight of these conditions. There has been little opportunity for any one to develop an appreciable background of experience, and no exhaustive studies have been made of the applicability of accepted methods of measurement to the particular conditions existing in low-head installations. At least three methods have been applied with more or less satisfactory results to low-head installations, only one of which, the two-type current-meter method, was developed with the operating conditions peculiar to low-head installations in mind; yet, this case is admittedly one of the most difficult of water-measurement problems, even when ordinary accuracy is desired. One may well question the ability of any method not primarily designed for operation under the conditions obtaining in low-head installations to give results whose accuracy would consistently approach a fraction of 1%. In the interest of future application, and in view of the difficulties attendant on a more precise procedure, it would seem worth while to compare the various methods of measurement existent before arbitrarily attributing to certain of the methods advantages which they may not possess, or limits of accuracy which careful investigation and study may show to be unattainable. Until such information is available, logic would dictate the acceptance of the method specifically developed for application to low-head installations as being preferable to other methods whose suitability in this particular case has not been rigorously demonstrated.

In this connection the unsoundness of judging the accuracy of a measurement method by comparison with expected turbine efficiencies calculated from model tests must be evident. Professor Moody's remark in the paper presenting his formula for efficiency step-up—"Considering the fact that the losses in a turbine are dependent on many factors and involve many complications and that in comparing efficiencies only approximate results at best can be expected * * *" [29]—might well be recalled here.

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[28] "Code d'Essais des Installations Hydrauliques," 1re ed., *Société Hydrotechnique de France*, pp. 46, 47.

As noted by Mr. Davis, the success of the Allen and Gibson methods is in part due to the strict control over their use exercised by Messrs. Allen and Gibson, who have found the thorough training of men in the use of their methods to be necessary. It is worth noting that in Europe neither of these methods is commonly accepted as a standard of measurement; the Test Code of the Société Hydrotechnique de France, for example, lists [28] them both in the class of methods whose "results appear uncertain either by reason of their principle of operation, insufficient development, or difficulties of practical application."

It appears that the profession needs to realize that, when high accuracy is required, water measurement is a specialized problem that may be handled successfully only by men having a background of knowledge and experience, using measurement methods whose applicability to the conditions of test has been proved. The observation of Mr. Davis regarding the necessity for trained engineers in the conduct of current-meter tests thus does not introduce a new requirement peculiar to this method of measurement, and the method should not be criticized on this account. In fact, there may be evidence of unsuccessful applications of accepted test code methods which would lend weight to the argument that these methods are at least as susceptible as the two-type current-meter method to difficulties traceable to inexperienced operators.

The unquestioned necessity for accurate and complete comparisons of model and prototype tests of turbine installations gives an increased importance to field testing of turbines. The consequent need for a clarification of the present unsatisfactory status of the water-measurement problem, particularly in regard to low-head installations, demands positive collaborative action on the part of both turbine manufacturers and purchasers or users. Comparative tests of water-measurement methods to be made at the Santee-Cooper project on the suggestion of the Hydraulic Prime Movers Committee of the A. S. M. E. should be considered as only the first step in such a clarification.

E. Shaw Cole,[30] **Assoc. M. Am. Soc. C. E. (by letter).**—Under the heading of "Field Tests" Mr. Davis mentions the various methods for measuring water and reviews the extensive research on the "multiple-current-meter method." It is the purpose of this discussion to bring attention to the equally extensive research on the pitot tube during the ten years 1929–1939.

Three papers since 1935 have presented the results of research at the Alden Hydraulic Laboratory and the University of Pennsylvania, on both the so-called simple and combined types of pitot tube. Two of these papers[31] were chiefly concerned with a combined type of tube that has been used successfully for accurate water measurements for more than forty years. Simultaneous measurements of turbine efficiencies using the pitometer and salt-velocity methods were presented and the close agreement was further proof that pitot tube measurements could be made with an error of less than 1%. The third paper[32] presented the results of a very thorough investigation of sources of error in pitot tubes. Various types of simple and combined tubes were studied, and one of the conclusions reached by Professor Hubbard was that the pitometer was a very convenient and accurate instrument.

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[29] "The Propeller Type Turbine," by Lewis F. Moody, *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 626.

[30] Engr., The Pitometer Co., New York, N. Y.

[31] "Pitot-Tube Practice," by Edward S. Cole, M. Am. Soc. C. E., *Transactions*, A. S. M. E., Vol. 57, 1935, p. 281; also "Pitot Tubes in Large Pipes," by E. S. Cole and E. Shaw Cole, *loc. cit.*, August, 1939, p. 465.

Mr. Davis' statement, under the heading "Field Tests," that the pitot tube method becomes "unwieldy when the size of the conduit passes medium size and for medium or large volumes of water" is vague and somewhat misleading. All methods of water measurement have limitations. For the pitot tube the size of conduit and velocity of flow may introduce mechanical difficulties, but for most cases these can be overcome by proper design of the pitometer rod and means for handling it. Such measurements have been made without difficulty in pipes 12 ft in diameter where the velocity was 9 ft per sec, and in 6-ft pipes where the velocity was as high as 19 ft per sec. Regular present-day flow measurements in pipes and on ships cover a range of velocities from 0.7 to 50 ft per sec, and although an extreme combination of large diameter with high velocity could present difficulties this would be unusual. For most cases the advantages of the pitot tube method (low cost and speed) are available to the profession without a sacrifice of accuracy.

The writer heartily agrees that strict control of the various methods of water measurement is highly desirable and would like to emphasize that the pitot tube method also should be applied only by men trained and experienced in its use.

Paul L. Heslop,[33] M. Am. Soc. C. E. (by letter).—Model testing has been used extensively as a means of advancing the art of turbine building, but it also has an immediate and practical side. When a customer buys a turbine he wants to be sure that it will perform as anticipated, and model testing has now reached the stage of reliability where reasonably accurate forecasts of performance should be made. Several late government contracts for large wheels, of which Bonneville is an example, have specified that acceptance shall be based on stepped-up model tests. This applies particularly to large low-head developments where accurate measurement of flow is difficult for the reasons stated by Mr. Davis.

At Bonneville a very thorough field test was made on the 81-in. Kaplan station-service unit in order to prove or disprove the reliability of the forecasting value of model tests. The tests show that, throughout the entire range of performance over a wide variation of load and of head, actual results do not differ more than 2% from values that could be predicted and, in the vicinity of contract head and rating, the agreement is a matter of tenths of a per cent.

With the field data at hand, some paper experimenting was done with step-up procedures. An expectation curve is shown in Fig. 22. To derive this curve the model efficiencies were stepped up by Moody differentials. The power was first stepped up by the relation of three-halves power of the diameters and then further increased by application of the same Moody differentials. The agreement around specified head and power is interesting. In computing the Moody differentials, the peak efficiencies of the model without respect to speed were used, which is the conventional method. Had the speed at 1-ft head of the model corresponding to actual prototype speed been used for efficiencies, the agreement between predicted and field tests would have been even closer.

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[32] "Investigation of Errors of Pitot Tubes," by C. W. Hubbard, *Transactions*, A. S. M. E., August, 1939, p. 477.

[33] Empresas Electricas Brasileiras, S. A., Rio de Janeiro, Brazil.

One of the means used on the Bonneville runners to combat cavitation was to cut out, and pre-weld with stainless steel, certain blade areas most susceptible to cavitation. In making cavitation experiments on a composite blade of this kind, it should be borne in mind that paint will not adhere as well to stainless steel as to cast steel.

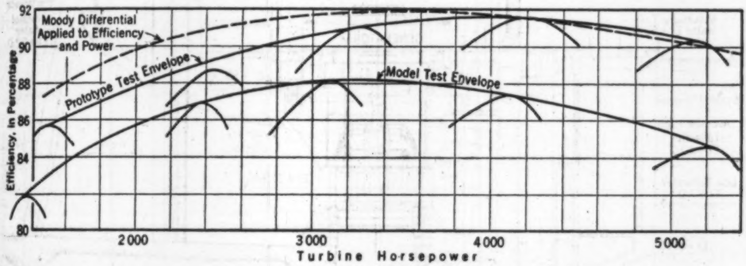

Fig. 22.—Bonneville Station-Service Turbine; 81-In. Kaplan Type Turbine Under a 50-Ft Contract Head

One purpose of the transparent models used in the Bonneville experiments was to decide upon a workable scroll case in the shortest possible time. When Bonneville was authorized, it was desired to put the greatest number of men to work in the shortest possible time. Immediate work on power-house construction could further this aim. At a later date, cavitation and performance tests were run on models by the turbine manufacturer. Time permitting, perhaps the ideal situation would be to combine visual analysis with the performance and cavitation model tests, using transparent sides for model make-up.

J. D. Scoville,[34] Esq. (by letter).—Mr. Winter discusses the tendency toward the use of a horizontal splitter in draft tubes. Fig. 23 is a section through the Bonneville power house showing the shape of the intake, scroll, and draft tube. The horizontal splitter shown gave a substantial improvement in draft tube performance as compared with tubes without splitters when tested with a model turbine. This type of tube was used also on the station service unit at Bonneville and partly explains the excellent performance of the turbine, which is shown in Fig. 3.

Scroll.—Model tests indicate that a simple design of scroll is superior to that which was used in the Bonneville power house, the plan of which is shown in Fig. 24. It will be noted that there is a multiplicity of piers and islands, the purpose of which is to guide the water into the turbine without the formation of eddies. It would appear that the additional skin friction introduced by the extra pier area more than compensates for any possible reduction of

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[34] Asst. Chf. Engr., S. Morgan Smith Co., York, Pa.

eddy loss. Fig. 25 shows the comparison of model tests on the U. S. Engineer's design of scroll in conjunction with a 16-in. model runner homologous to the Bonneville turbine. These comparisons are made at model speeds correspond-

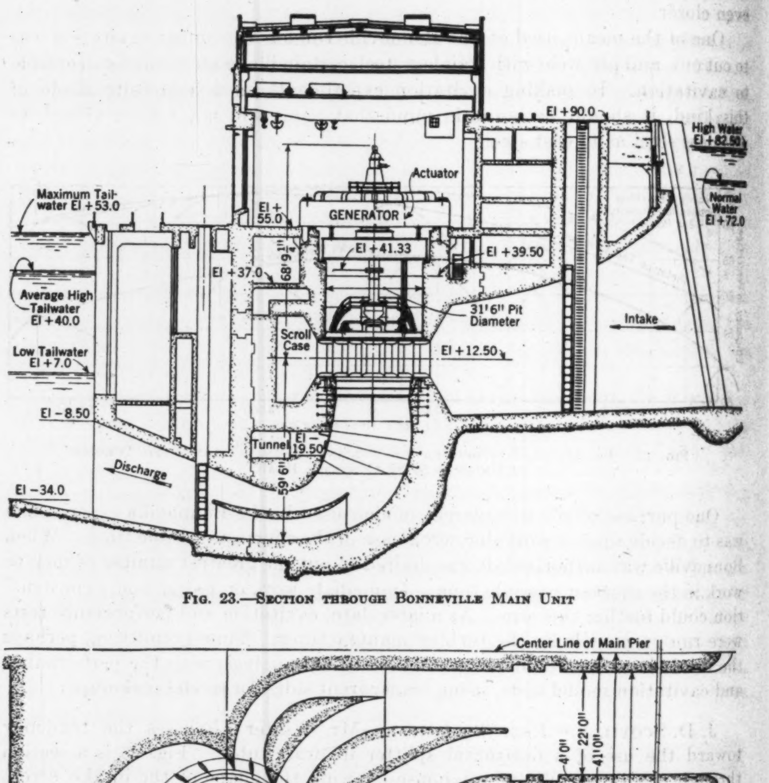

El +90.0
High Water El +82.50
Normal Water El +72.0
Actuator
Maximum Tailwater El +53.0
El + 55.0
GENERATOR
El +41.33
El +39.50
El +37.0
68'9¼''
Average High Tailwater El +40.0
31'6'' Pit Diameter
Intake
Scroll Case
El +12.50
Low Tailwater El +7.0
El -8.50
Tunnel
El -19.50
Discharge
59'6''
El -34.0

FIG. 23.—SECTION THROUGH BONNEVILLE MAIN UNIT

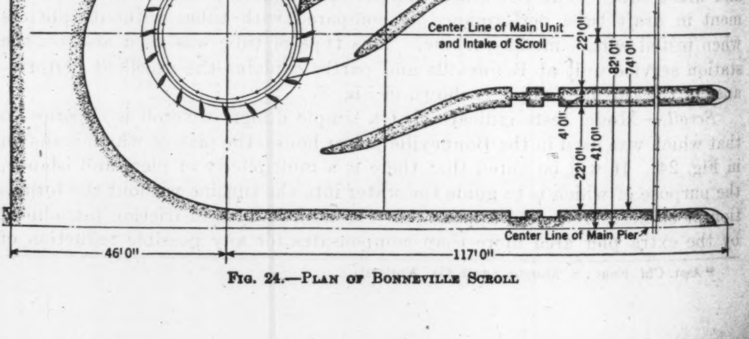
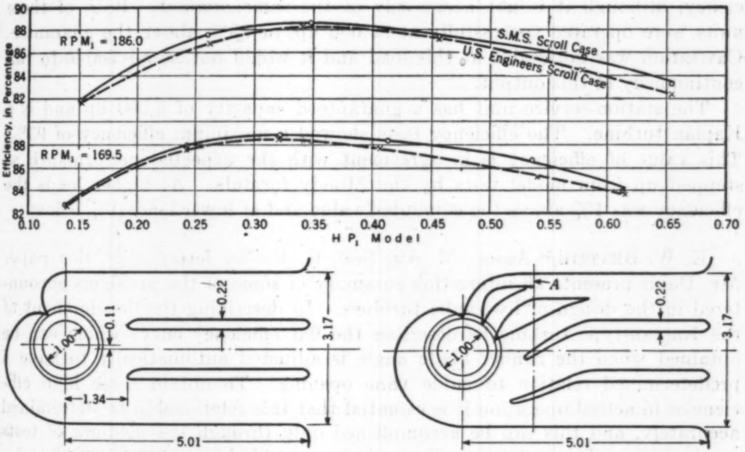

Center Line of Main Pier
22'0''
41'0''
4'0''
Center Line of Main Unit and Intake of Scroll
22'0''
82'0''
74'0''
4'0''
41'0''
22'0''
Center Line of Main Pier
46'0''
117'0''

FIG. 24.—PLAN OF BONNEVILLE SCROLL

ing to 50-ft and 60-ft heads and show that the simple design of scroll is superior to the more complicated one which was used. Structural considerations made it necessary to use the U. S. Engineer's design. Where it is not necessary to carry the intermediate piers too close to the speed ring of the turbine, the simpler design is preferable.

***Scroll Velocities.*—Mr. Winter states that it is possible to use considerably higher scroll velocities with improved turbine performance. The writer has tested plate steel scrolls with velocities 60% above the normal value and found a loss in efficiency of about 1%. This would indicate the possibility of using higher scroll velocities. However, he questions the advisability of going to so high a velocity because, although there may be little loss when the unit is new, the loss may be substantially greater when the unit is several years old and the scroll is roughened from corrosion.**



S. Morgan Smith Design of Scroll Case

U.S. Engineers Design of Scroll Case

FIG. 25.—COMPARISON OF MODEL SCROLL TESTS

***Air Admission.*—Mr. Winter refers to the admission of air through the hollow bore in the turbine shaft at Boulder Dam and Drop No. 4 for the purpose of eliminating shock in the draft tube caused by vortex cavitation; and he states that this was found to have no adverse effect on efficiency or horsepower. This is not always true. The writer has had opportunity to observe the effect of air admission on efficiency on three plants. The unintentional admission of air through the hollow shaft of a 53,000-hp unit under a 135-ft head caused a loss in efficiency of 3%. Experimental air admission to a 42,000-hp turbine under a 104-ft head caused an efficiency loss of 3% at the peak and a gain of 1% at 25% load. Air leakage through the packing gland of a 3,000-hp turbine under a 54-ft head produced an efficiency loss of 4.5%. The advisability of admitting air must be determined for each installation.**

Moody Differential.—It is the writer's experience that he obtains values higher than the computed Moody differential on the smaller sizes of runners, to a diameter of 100 in. Beyond this diameter there is some indication that the Moody step-up ratio is a little too high; but this possibility is supported by very meager data. The writer agrees with Mr. Davis that more tests on very large units are desirable. Only by numerous points on the curve can it be determined whether the Moody exponent should be 0.25 or 0.20.

Bonneville Tests.—In 1938 power tests were made on the main units at Bonneville, and efficiency tests were made on the service unit. The main units, two in number, have a rating of 60,000 hp under a 50-ft head. Before the job was built, complete model tests were made in the manufacturer's laboratory and at Holtwood, Pa. Field tests at four heads, from 29 to 53 ft, showed about 8% more power than that proportioned from the model tests. This increase in power is certainly evidence of a substantial increase in efficiency, although it is not necessarily of the same amount. Both of these units were operated successfully at 80,000 hp, or 33% above the guarantee. Cavitation was noticeable at this load and it would not be advisable to run continuously at this output.

The station-service unit has a guaranteed capacity of 5,000 hp and is a Kaplan turbine. The efficiency tests showed a maximum efficiency of 93%. This value of efficiency is in agreement with the expected performance as stepped up from model tests by the Moody formula. At higher loads the efficiency was 1% above the computed value and at lower loads 1% below.

K. W. BEATTIE,[35] ASSOC. M. AM. SOC. C. E. (by letter).—In this paper Mr. Davis presents an interesting summary of some of the problems encountered in the design of hydraulic turbines. In describing the development of the Kaplan-type turbine he describes the flat efficiency curve which can be obtained when the runner blade angle is adjusted automatically to have a predetermined relation to guide vane opening. To obtain these high efficiencies in actual operation it is essential that this relationship be determined accurately, and this can be accomplished only through the medium of tests conducted on the prototype. Since the correct blade-gate relation differs for various effective heads, such tests should be repeated at different operating heads. These tests on the prototype are easily conducted, using casing piezometers as an index of flow, and should not be neglected if serious losses in operating efficiency caused by improper setting of the runner blades are to be avoided.

Discussing the subject of cavitation testing, Mr. Davis states that tests at the Holtwood laboratory, and others by Professor Spannhake, have indicated that there is no variation in final test results when the test head is varied from 58 ft to less than 33 ft. Tests in the I. P. Morris laboratory demonstrate that cavitation tests may safely be conducted at heads as low as 25 ft, provided that excessive quantities of air are not present in the water. When this laboratory was put in operation for cavitation tests on 11-in. diameter model turbines, the first test to be conducted was on a model that was very carefully machined to be exactly homologous with a 16-in. diameter unit previously tested at the Holtwood laboratory. Fig. 26 shows some of the sigma curves from this test plotted on the same sheet with similar curves from the Holtwood test. In spite of the difference in test heads from 39 ft to 25 ft, and the difference in the size of units, an excellent agreement is indicated in location of the breaks in the sigma curves.

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[35] Research and Test Engr., Baldwin-Southwark Corporation, I. P. Morris Div., Eddystone, Pa.

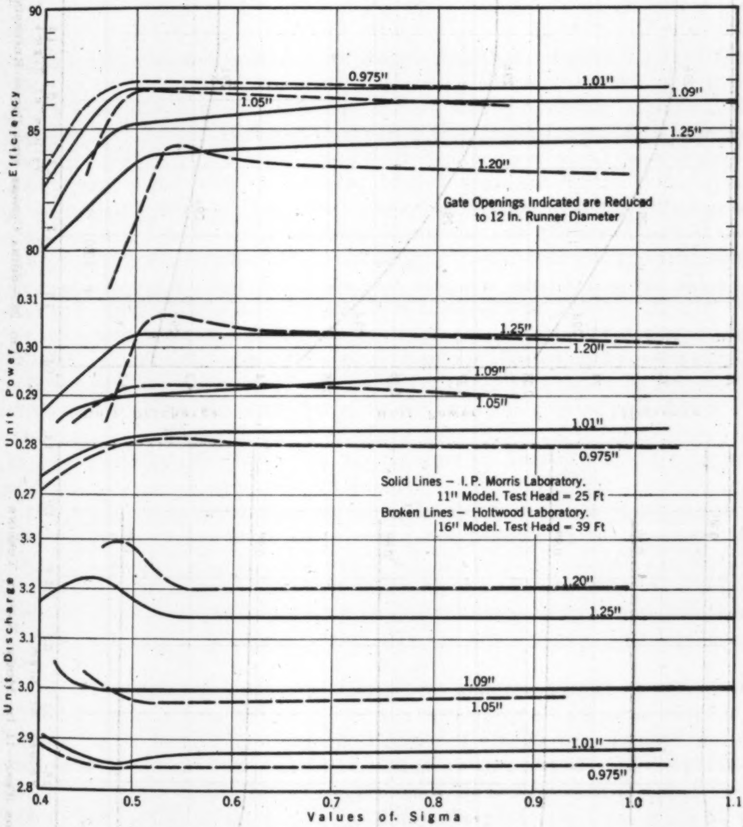

FIG. 26.—COMPARISON OF CAVITATION TESTS OF HOMOLOGOUS RUNNERS MADE UNDER DIFFERENT TEST CONDITIONS

In regard to the shape of sigma curves, Mr. Davis states that discharge is increased when cavitation occurs, whereas results of tests of some model runners have shown that the discharge may decrease slightly when cavitation first starts. Tests of a number of different runners made under a wide range of conditions of blade angle and phi have resulted in sigma curves of many

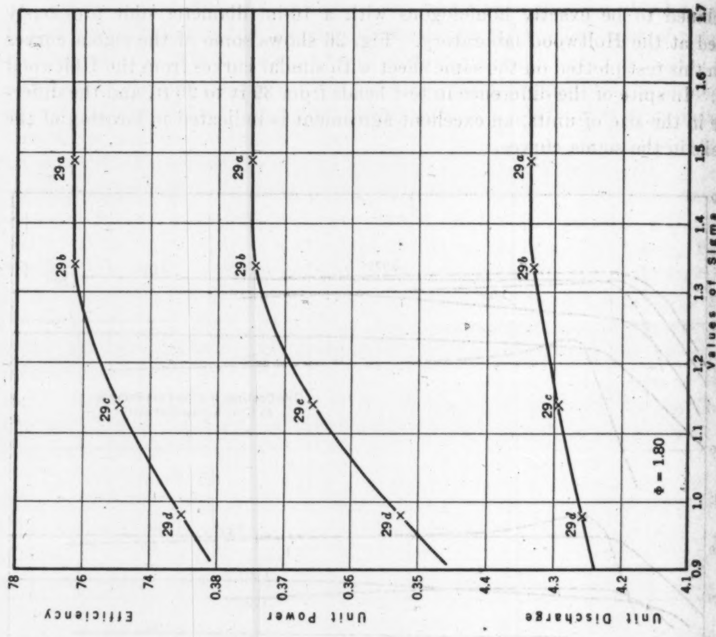

FIG. 28.—SIGMA CURVES WITH POINTS REFERRING TO FIG. 29

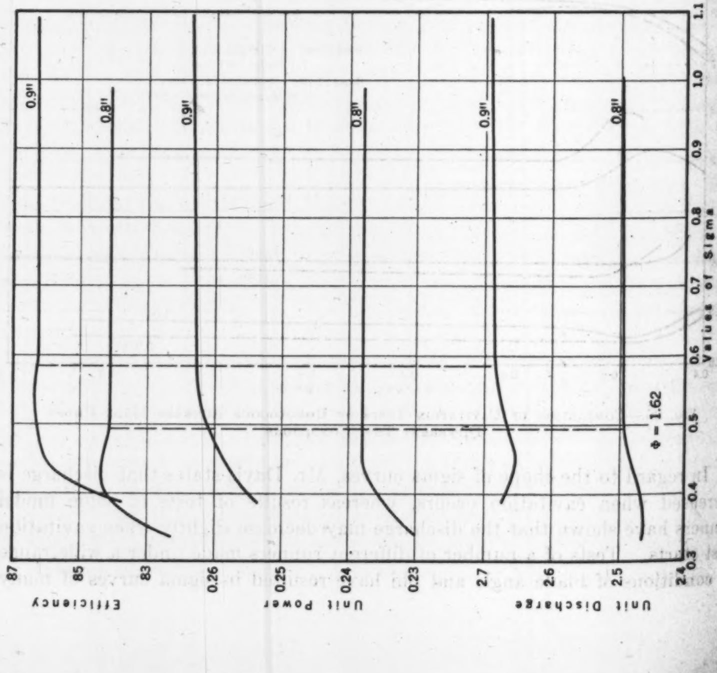

FIG. 27.—CAVITATION TEST—11-IN. MODEL KAPLAN TURBINE

different shapes, some of which are as indicated in Fig. 16 of the paper. Figs. 27 and 28 show some other curves, and in these the discharge decreases. In the curves of Fig. 27, the decrease in discharge is not immediately accompanied by a drop in power; so there is a slight rise in efficiency, at low values of sigma. In some cases, this efficiency increase has amounted to several per cent. In a few cases an increase in power has occurred over a narrow range of sigma, near the break.

It will be noted that the curves of Fig. 27 do not show a coincidence of the breaks of the discharge, power, and efficiency curves, and in some cases the spread between these breaks has been greater than that indicated herein. The writer cannot agree that this, of itself, is evidence of faulty design, nor that such a conclusion is the consensus of opinion among hydraulic engineers, as stated by Mr. Davis. It should be expected that cavitation will first occur at some local portion of the blade where the pressure is a minimum and progressively spread as the pressure is reduced, with an accompanying gradual change in turbine performance. To expect general cavitation over the entire area simultaneously is to assume uniformity of pressure over the entire back of the blade, and this is not essential to good design. In the case of Kaplan runners designed to operate efficiently under wide ranges of head and power output, such uniformity of pressure is impossible of achievement.

The criterion which differentiates between good and faulty design is the value of sigma at which the turbine performance is affected, and not the shape of the curves. As stated by Mr. Davis, the critical sigma should be defined as that value at which the first change in performance is noted, whether it is manifested by a change in power or discharge, and even if it is accompanied by an increase in efficiency. The broken vertical lines of Fig. 27 indicate critical sigmas according to this definition.

Very interesting visual observations of the occurrence of cavitation on the backs of model runner blades have been made possible by the provision of windows in the bottom of a model draft tube in conjunction with a powerful stroboscopic light source. The light flashes are synchronized with the turbine shaft, and the source is a vapor lamp producing flashes of very short duration so that the runner is completely "stopped" and photographic time exposures can be made.

Fig. 29 contains photographs of an 11-in. diameter runner operating at 1,600 rpm under a head of 28 to 30 ft, and developing a unit power of 0.375, the value of phi being 1.80. The sigma curves of Fig. 28 were obtained under these same conditions and indicate a break at a sigma of 1.37. In the photograph, light areas on the blades are produced by the presence of vapor and indicate the cavitation areas. In Fig. 29(*a*), the photograph was made at a sigma of 1.49 and small areas of cavitation are discernible at the periphery and close to the hub, although the turbine performance has not been affected. The other photographs are at successively lower sigmas and the progressive spread of the cavitation areas is apparent.

The photographs shown were selected for this discussion because of their clarity rather than for excellence of turbine performance or because they were considered typical. The pattern of cavitation areas is quite different for various runners and different operating conditions. Photographic records of cavitation areas provide information regarding the location of areas on the

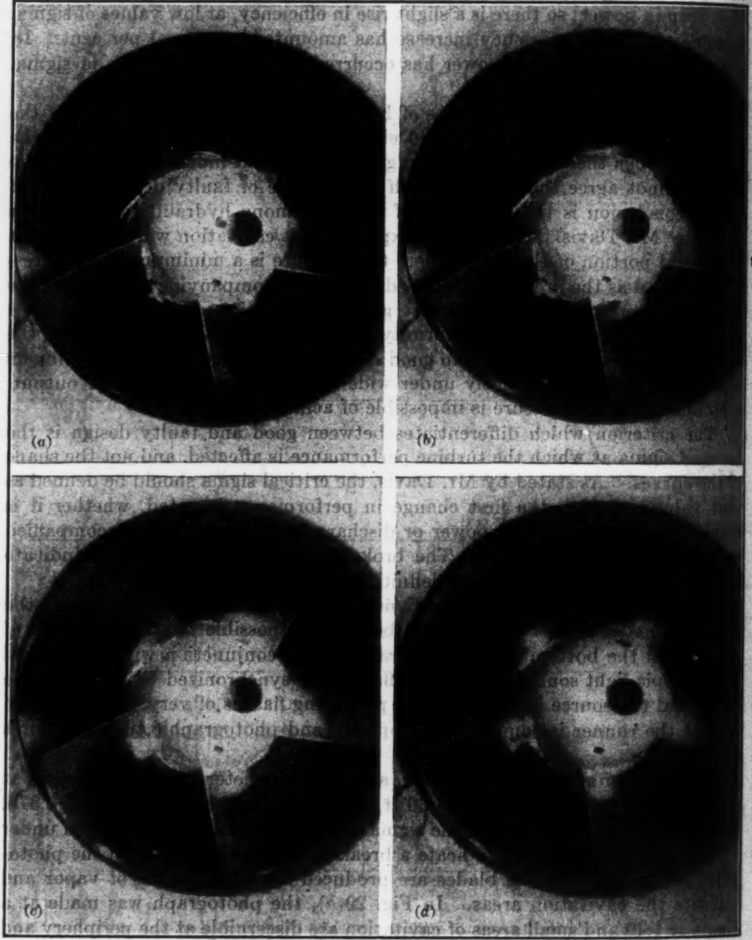

FIG. 29.—RUNNER, 11 IN. IN DIAMETER, OPERATING AT 1,600 RPM, UNDER A HEAD OF 28 TO 30 FT AND DEVELOPING A UNIT POWER OF 0.375 (ϕ = 1.80)

prototype to be prewelded with stainless steel, and also furnish valuable clues to improvements in design.

Mr. Davis stresses the importance of conducting cavitation tests on completely homologous units. It is agreed that this is an important point as regards the wheel setting and draft tube, but the necessity of including the intake and casing has not been established. It is doubtful that any disturbance produced in a casing of usual design would have an effect upon cavitation in the runner. One group of tests has been made in the I. P. Morris laboratory during which the same unit was tested with and without the model casing. There was no difference in the appearance of the sigma curves.

A procedure for relating the results of cavitation tests on the model turbine to performance of the prototype has been employed which differs slightly from that described by Mr. Davis. Critical values of sigma are plotted against unit power, and it is assumed that cavitation may occur at the same unit power on the prototype. Such a curve is shown in Fig. 30, plotted on the same sheet with efficiency. Because of the increased efficiency of the large unit, this method results in a more conservative interpretation than does the method described by Mr. Davis in which unit discharge is the constant.

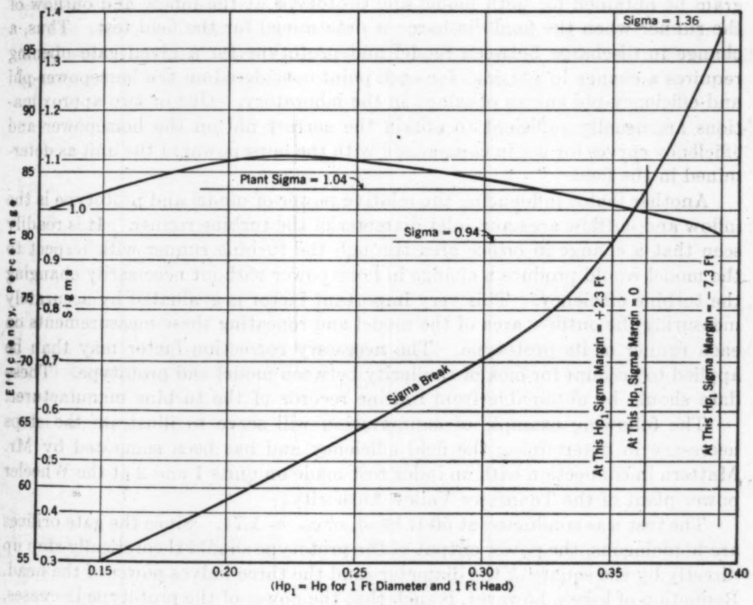

FIG. 30

Referring again to Fig. 30, it will be noted that the curve of critical sigma rises sharply as the maximum power is approached. This demonstrates the importance of limiting the power output of the unit to the value corresponding to the plant sigma. Increasing the output beyond this point involves operation at sigmas well below the critical value and may result in serious damage due to cavitation pitting.

There can be nothing but agreement with Mr. Davis when he suggests that additional study and laboratory work will result in further improvements in performance of hydraulic turbines. The investigation of cavitation characteristics alone offers an extensive field for further research.

I. A. Winter,[36] Assoc. M. Am. Soc. C. E. (by letter).—Mr. Mattern has suggested that a brief description be given of the power step-up method which the writer has proposed for predicting the performance of prototype turbine installations from test results on strictly homologous models. The method involves a comparison of power output between model and prototype and in this respect offers a comprehensive means for predicting field efficiency. Basically, the method requires an increase in efficiency for an increase of horsepower of prototype over model and a decrease in efficiency for a decrease in horsepower.

The proposed method further requires that a facsimile velocity-vector diagram be obtained for both model and prototype at the inflow and outflow of the runner when the final discharge is determined for the field test. Thus, a change in discharge between model and prototype for a given gate opening requires a change in phi (ϕ_m) for each point considered on the horsepower-phi and efficiency-phi curves obtained in the laboratory. One or two approximations are usually sufficient to obtain the correct phi on the horsepower and efficiency curves for use in comparison with the horsepower of the unit as determined in the field.

Another factor influencing the relative power of model and prototype is the inflow and outflow area and inlet diameter of the turbine runner. It is readily seen that a change in orifice area through the turbine runner with respect to the model would produce a change in horsepower without necessarily changing the turbine efficiency. This very important factor is evaluated by accurately measuring the outflow area of the model and repeating these measurements on each runner of its prototype. The necessary correction factor may than be applied to account for lack of similarity between model and prototype. These data should be obtainable from routine records of the turbine manufacturer.

The following example of computation will serve to illustrate the steps necessary in determining the field efficiency and has been suggested by Mr. Mattern in connection with an index test made on units 1 and 2 at the Wheeler power plant of the Tennessee Valley Authority:

The test was conducted at 50-ft head, or $\phi_m = 1.74$. Since the gate orifices are homologous, the power output of the prototype should theoretically step up directly by the square of the diameters and the three-halves power of the head. Reduction of losses, however, is such that the power of the prototype increases, and the actual phi at which the runner is operating must have changed. The step-up method begins by first taking this into account in Equation (1); thus:

$$\phi = \frac{p_m}{P}\phi_m \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (20)$$

in which: ϕ_m = phi, or the ratio of peripheral velocity of runner to spouting velocity of water at the head used for the field test; ϕ = effective phi at which runner is operating; p_m = field power stepped up directly from model performance at ϕ_m; and P = prototype power. For $\phi_m = 1.74$, Equation (20) becomes $\phi = \frac{47{,}000}{48{,}000} \times 1.74 = 1.704$ (first approximation).

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[36] Senior Engr., U. S. Bureau of Reclamation, Denver, Colo.

Going to the model test data and picking off the values for the proper gate opening, it is found that the field power p_ϕ, stepped up from the model performance at ϕ, = 46,650, and the model efficiency (e) at ϕ = 87.2%. The discharge can then be computed:

$$q = \frac{550\, p_\phi}{62.5\, h\, e} \quad\text{(21)}$$

in which: q = discharge computed from model efficiency and stepped up power at ϕ; and h = head required to produce stepped up model power at 100% model efficiency (e), using p_ϕ and e. From Equation (21), $q = \frac{550 \times 46{,}650}{62.5 \times 50 \times 0.872}$ = 9,416 cu ft per sec.

The theoretical head required to produce the foregoing power is then determined. By this means it is possible to work directly with the fundamental quantities without adjusting for variable losses; thus:

$$h = \frac{550\, p_\phi}{62.5\, q} \quad\text{(22)}$$

From Equation (22), $h = \frac{46{,}650 \times 550}{9{,}430 \times 62.5} = 43.60$ ft.

Outputs are proportional to the three-halves power of the heads so that $\frac{p_\phi}{h^{1.5}} = \frac{P}{H^{1.5}}$; from which,

$$H = \frac{h\, P^{0.67}}{p_\phi^{\,0.67}} \quad\text{(23)}$$

From Equation (23), $H = \frac{43.6 \times (48{,}000)^{0.67}}{46{,}650^{0.67}} = 44.44$.

Discharge is proportional to the square root of the heads so that

$$Q = \frac{q\sqrt{H}}{\sqrt{h}} \quad\text{(24)}$$

in which Q = discharge of prototype. From Equation (24), $Q = 9{,}416 \times \frac{\sqrt{44.44}}{\sqrt{43.60}} = 9{,}506$ cu ft per sec. The final value desired is the prototype efficiency E, and this can be computed from the fundamental formula

$$E = \frac{550\, P}{62.5\, H_T\, Q} \quad\text{(25)}$$

From Equation (25), $E = \frac{48{,}000 \times 550}{50 \times 9{,}506 \times 62.5} = 88.8\%$ (based on first approximation). The corresponding result from the Moody formula would have been

$$E = 100 - (100 - e)\left(\frac{d_m}{D}\right)^{0.25}\left(\frac{h_m}{H_T}\right)^{0.01} \quad \ldots\ldots\ldots\ldots (26)$$

From Equation (26), $E = 100 - (100 - 87.2)\left(\frac{16}{264}\right)^{0.25}\left(\frac{3.8}{50}\right)^{0.01} = 93.81\%$.

The preceding example indicates the greatly different results obtained by the power step-up method and the formula proposed by Professor Moody. The performance curve shown in Fig. 31 for units 1 and 2 of the Wheeler power plant has been derived by the power step-up method based on comprehensive index tests fully equal to an acceptance test, in regard to the number of observations made and in degree of accuracy of measurement and computation. These curves may be considered as very satisfactory.

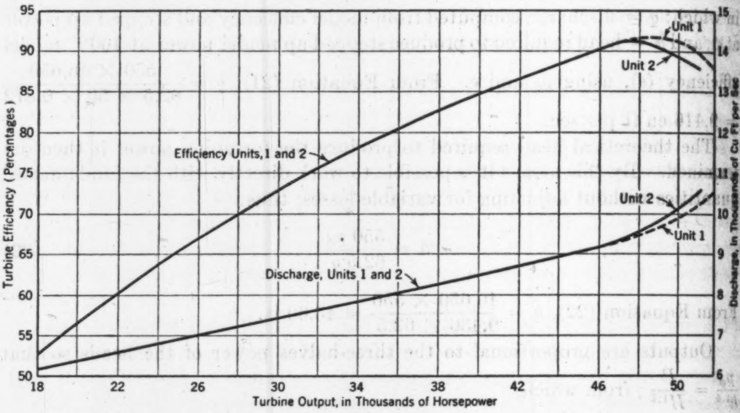

FIG. 31.—TURBINE EFFICIENCY CURVES—UNITS 1 AND 2 AT 49 FT NET TURBINE HEAD

Where a hydraulic laboratory has established a definite efficiency-increase ratio between laboratory and field tests, based on years of experience, no necessity is seen for the use of a step-up formula, since in the final analysis the laboratory differential must be fulfilled if there is sufficient similarity between model and prototype. It is not conceivable that any manufacturer would fail to meet guaranteed efficiency where final acceptance of the turbine is made on the basis of tests on laboratory models.

However, results of field tests do not confirm full attainment in all cases of the laboratory and field differential, and hence the power step-up method is proposed, since it definitely places responsibility for fulfillment of field requirements without the necessity of a greatly involved testing procedure, results of which might be questionable due to unfavorable flow measuring conditions.

In the first paragraph of his discussion, and in Fig 20, Mr. Sharp presents data comparing the performance of a runner model, 11 in. in diameter, tested in a spherical shaped throat ring, with its performance tested in a cylindrical throat ring. These data are at variance with the test results on which the writer based his statements in the last paragraph under "Turbines."

The latter test results were made on a five-blade adjustable runner with a nominal diameter D of 16.5 in., under about 12 ft of head, in the hydraulic laboratory at Newport News, Va. In Fig. 32 comparisons of enveloping efficiency against unit power are made between a cylindrical and spherical throat ring at unit speeds of 160, 180, and 200. The corresponding values of peripheral coefficient (ϕ) are 1.44, 1.62, and 1.80 and are about the same as those noted in Fig. 20. The test results indicate a definite advantage of the cylindrical throat over that particular spherical throat ring. The smallest diameter of the spherical throat ring was 16.00 in., or 0.970 D, as compared with Mr. Sharp's value of 0.983 D. Since the burden on the draft tube varies inversely as the fourth power of the throat diameter, the 16-in. spherical throat places a burden on the draft tube that is about 12.5% greater than that of the 16½-in. cylindrical throat. Consequently, it should not be surprising that the cylindrical throat shows better performance at the higher discharges. It is true that the greater clearance at the higher blade angles causes some greater leakage loss. However, that may be overbalanced by the eddy loss created downstream from the somewhat abrupt curvature of the spherical shape.

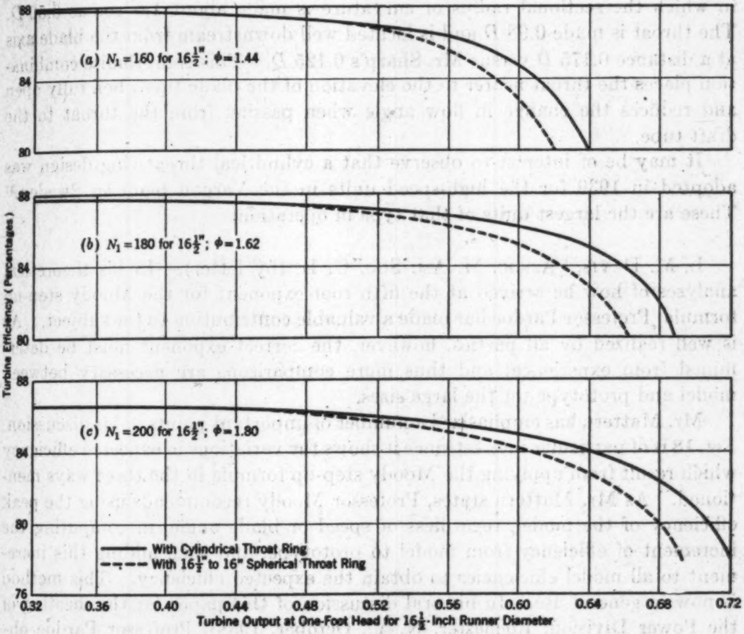

FIG. 32.—COMPARISON OF MODEL TEST RESULTS ON 16½-IN. DIAMETER, FIVE-BLADE ADJUSTABLE RUNNER

The test results at Newport News, in general, indicate that with the adjustable runners of higher specific speed there is an advantage of the cylindrical over the spherical throat, whereas in the case of the lower specific speeds there is not much difference. For the lower specific speeds a shape has been adopted, that is intermediate between the cylindrical and spherical shapes, in which the sectional radius of curvature is made about 1.6 versus 0.50 *D*. The throat is made 0.98 *D* and is located well downstream from the blade axis at a distance 0.175 *D* versus Mr. Sharp's 0.125 *D*. This compromise combination places the throat nearer to the elevation of the blade tips when fully open and reduces the change in flow angle when passing from the throat to the draft tube.

It may be of interest to observe that a cylindrical throat ring design was adopted in 1939 for the high-speed units in the Vargon plant in Sweden.[37] These are the largest units of that type in operation.

L. M. Davis,[38] Assoc. M. Am. Soc. C. E. (by letter).—In his theoretical analyses of how he arrived at the fifth root exponent for the Moody step-up formula, Professor Pardoe has made a valuable contribution to the subject. As is well realized by all parties, however, the correct exponent must be determined from experience; and thus more comparisons are necessary between model and prototype for the large sizes.

Mr. Mattern has emphasized a number of important points in his discussion. Fig. 18 is of particular interest since it shows the variations in expected efficiency which result from applying the Moody step-up formula in the three ways mentioned. As Mr. Mattern states, Professor Moody recommends using the peak efficiency of the model, regardless of speed or blade angle, in computing the increment of efficiency from model to prototype, and then adding this increment to all model efficiencies to obtain the expected efficiency. This method is now in general use. In his oral discussion of this paper (at the meeting of the Power Division, Rochester, N. Y., October, 1938), Professor Pardoe objected to this method since at zero power it would result in an efficiency equal to the increment, which, of course, is impossible. He suggested that a more rational method of applying the increment would be to multiply it by the ratio,

$$\frac{\text{Discharge at best efficiency for head and speed conditions in question}}{\text{Discharge corresponding to the efficiency point to be stepped up}}.$$

The result would then be added to the model efficiency. The tendency would then be to give lower expected efficiencies at all points below the point of best efficiency, and higher for all points above, than the usual method. This would be a step in the right direction because ordinarily the prototype efficiency has such a tendency in comparison to the model efficiency.

Professor Moody has shown that the exponent used in the Moody step-up formula was determined from comparative model and prototype tests available at that time. This will answer Mr. Mason's question of the advisability of using a theoretical formula which has a none too stable background.

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[37] *Power*, December, 1939, pp. 744–746.

[38] Hydr. Test Engr., Pennsylvania Water & Power Co., Holtwood, Pa.

The remarks regarding the limitations of the pitot tube to which Mr. Cole took exeption were, to say the least, vague. However, the idea in mind was that in a large unit intake having two or three intake passages of massive concrete it would be impracticable to attempt to measure water with a pitot tube, due to the mechanical difficulties and the expense involved.

Mr. Scoville states that on the 5,000-hp service unit at Bonneville the field test gave efficiencies 1% above the expected at higher loads than the best efficiency and 1% lower than the expected at lower loads. This adds weight to Professor Pardoe's suggestion for applying the step-up.

Mr. Beattie has presented some very interesting information based on experience. As he states, the behavior of the power and discharge characteristics of different turbine models is quite different near the point where cavitation starts. No doubt the location in the turbine where cavitation takes place dictates the behavior of the power and discharge characteristics. It seems quite logical that, if cavitation occurred well upstream from the point of smallest discharge area in the turbine, the discharge might be reduced, due to the turbulence set up by cavitation. If this reasoning is accepted as a cause for reduced discharge with the start of cavitation, it might be further reasoned that by a slight alteration in blade shape this low pressure zone occurring upstream from the point of maximum velocity might be eliminated and the cavitation characteristics thus improved.

The basis for the statement that failure of the unit power and unit discharge breaks to coincide is an indication of faulty design is that, in a turbine of perfect design, cavitation may be expected to occur first at the point of maximum velocity, which results in minimum pressure. This will naturally be at the minimum discharge area in the turbine where it seems logical to expect an effect on both power and discharge characteristics. However, if cavitation starts at one or more points on the blades away from this control point, the power may be effected without any change in discharge or the discharge may even be decreased as mentioned herein.

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WATER SUPPLY ON UPPER SALT RIVER, ARIZONA

BY JOHN GIRAND,[1] ASSOC. M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. DANA M. WOOD, LEROY K. SHERMAN, GEORGE F. MCEWEN, AND JOHN GIRAND.

SYNOPSIS

Presented in this paper are the data and methods used to determine economical storage requirements and power outputs of a proposed hydroelectric plant on the Upper Salt River, Arizona. The gaging station at the project site had only been in operation for a few years, necessitating a correlation of data which covered a longer period of time, including other stream-gage records, rainfall records, and tree-ring measures.

After the hydrograph of the river flows had been constructed, the storage requirements and power output of the plant were estimated by a new method involving the construction of a probable future hydrograph, based on laws of probable occurrence and well-defined trends of long-period variations in river flows. Due to the greater value of water in the semi-arid regions, particularly in periods of low flow, special treatment has been given to drought periods involving methods of probable sequence of drought years.

A copy of the paper, with two appendices containing detailed observations of runoff and rainfall, has been placed on file for reference at Engineering Societies Library in New York, N. Y.[2]

INTRODUCTION

The Salt River rises at an elevation of 10,000 ft above sea level in the White Mountains near the eastern boundary of the State of Arizona. It flows generally westward through heavily timbered plateau country with an average elevation of 6,000 ft until it reaches the Mogollon Rim, where it drops rapidly in a few miles to an elevation of 3,000 ft. Then it flows on a flatter grade through rolling and sparsely vegetated country into the Roosevelt reservoir near the central part of the state. The water impounded in the Roosevelt

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NOTE.—Published in December, 1939, *Proceedings*.

[1] Asst. Hydr. Engr., James B. Girand, Phoenix, Ariz.

[2] 33 West 39th Street, New York, N. Y.

reservoir is used for irrigation in the Salt River Valley (population, 100,000) and for the generation of power used in Phoenix and vicinity.

The relative isolation of the Salt River Valley results in high costs for fuels required for prime movers, and has lead to an intensive development of the hydroelectric resources of the state. Five hydroelectric plants are now (1939) operating on the Lower Salt River below the Roosevelt Dam with an installed generating capacity of 79,000 kw.

In spite of the intensive development of water resources, rapid increase in agricultural and industrial consumption of power has caused a shortage in the Salt River Valley. Heretofore, investigation of power possibilities on the Salt River above the Roosevelt Dam was not considered seriously, as good sites were available below Roosevelt Dam which would have the beneficial effect of regulation by storage above the power dams. At present, with the river completely regulated below the dam, as a matter of necessity the Upper Salt River was investigated.

In the canyon section, the Upper Salt River has a steep gradient and in places is extremely tortuous. In one place, Mule Hoof Bend, the river makes a 3-mile loop and returns within 400 ft of itself, with a 75-ft difference in elevation in water surfaces. At another place, Walnut Canyon, a conduit one mile long will cut off 3 miles of river and give a potential head of 300 ft. In this canyon section there are many excellent dam sites and power house locations; a main highway, constructed in 1937, gives easy access, and the project was seen at once to be capable of producing power very economically.

Due to the fact that there are a number of units which might be built economically, each operating under a different head, the water supply study was made on the basis of a unit head of 100 ft from which the output of any desired plant could be determined by multiplication by the proper head factor.

THE HYDROGRAPH

A gaging station has been maintained at the project site only since 1924. Below the project there are other gages which have been in operation intermittently since 1888. The locations of the various gaging stations are shown in Fig. 1.

Estimated values were determined from the direct relation between Chrysotile gage and other individual gages on the river, by plotting the simultaneous monthly values of the Chrysotile gage and the other gage under consideration. Logarithmic scales were used to keep the percentage of accuracy constant for all values of flows, as well as to allow determination of an equation of correlation in exponential form. From these graphs, the record of runoff at the Chrysotile gage was estimated for those years previous to 1924, which were covered by the Roosevelt and McDowell gages.

During 1889 and 1890 the only records obtained on the river were taken from observations at Arizona Dam, below the junction of the Salt and Verde rivers. The Verde River drains country with different topographic and climatic conditions; and the problem, first, of separating the flows at Arizona Dam into the Salt and Verde river components and, second, of reducing the Salt River component to a Chrysotile equivalent, introduced so many unknowns that with the short period of record available a satisfactory relation was not found.

Although it may be possible to determine a relation without too great an error, the accuracy of the data is questionable. The original observers' records show the discharges in cubic feet per second filled out daily with great care and decimal point exactitude at a time when Apache Indians were active in this area, leading to the conclusion that the flows were inserted by a not too conscientious observer. Furthermore, it is stated that the measurements were made in the canal headgates and that flood flows were only estimated and not measured. The data at hand allowed the construction of a hydrograph of estimated flows at the project site from January, 1891, to September, 1936 (see Fig. 2).

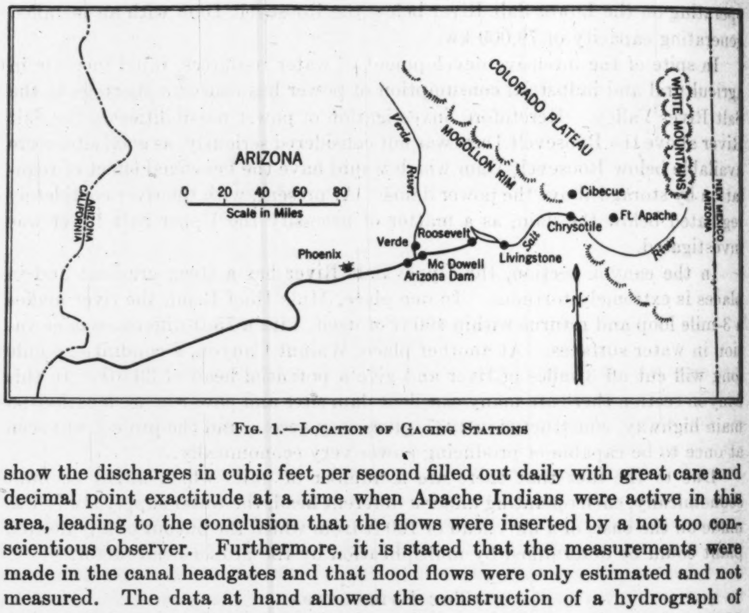
COLORADO PLATEAU
WHITE MOUNTAINS
Verde River
MOGOLLON RIM
ARIZONA
0 20 40 60 80
Scale in Miles
Cibecue
Ft. Apache
Chrysotile
Roosevelt
Verde
Phoenix
Mc Dowell
Arizona Dam
Livingstone
Salt River
ARIZONA CALIFORNIA
ARIZONA NEW MEXICO

FIG. 1.—LOCATION OF GAGING STATIONS

DATA DERIVED FROM THE HYDROGRAPH

In the foregoing, the data available were stated to be correlated to obtain a hydrograph of the estimated flow of the river at the project site during the forty-five years since 1891. With the hydrograph in Fig. 2 there were two general methods of analysis whereby an estimate of the future power output of the project might be obtained, either of which methods would not include an important factor in storage reservoir operation.

The mass graph, a summation of flows as they occurred in the past, treats the data in strictly chronological order. Probability curves and the probability derivative, the duration curve, treat the same data without respect to chronology. With either method, the important fact which is lacking is the sequence of flows to be expected.

Although it is well known that the mass graph treats the data in a sequence of flows as they occurred in the past, there is no evidence to indicate that the same sequence will be repeated in the future; even a superficial examination of the hydrograph will show that no sequence of flows covering a 2-year or a 3-year period has ever been repeated. Thus, an analysis based on a mass graph of past runoff, although admittedly giving a solution that would be most economical for a sequence of flows as they occurred in the past, would not necessarily give a solution that would be most economical for the sequence of flows that will occur in the future. In fact, as no evidence exists to show that 2-year or 3-year sequences have been repeated, it may be stated that a solution based on a mass graph of past runoff could not give a result which could be stated to be most economical for the years to come.

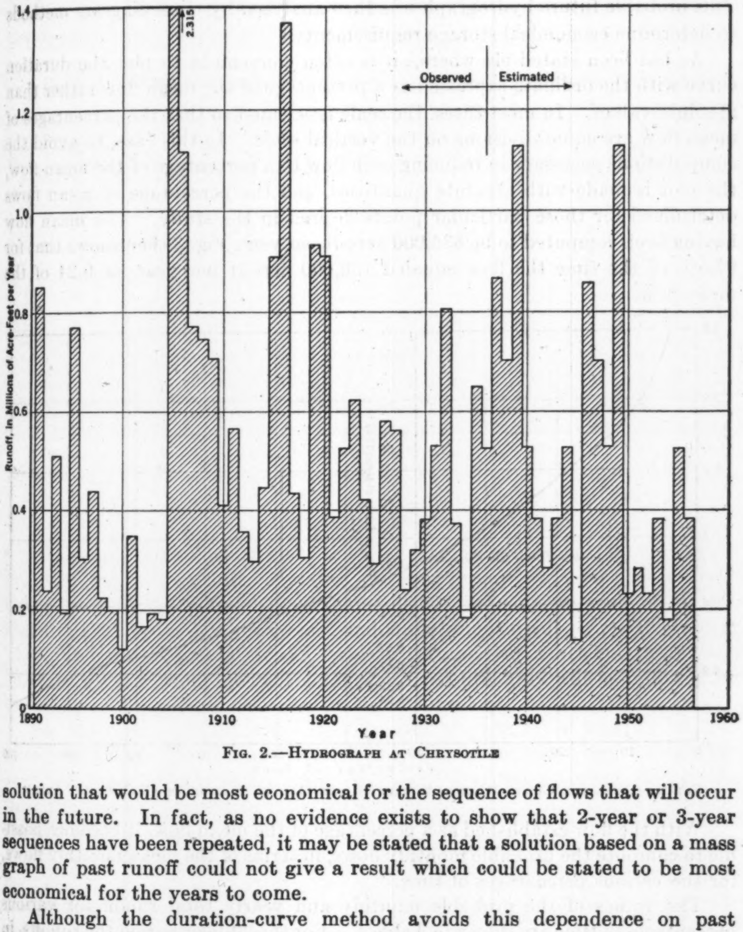

FIG. 2.—HYDROGRAPH AT CHRYSOTILE

Although the duration-curve method avoids this dependence on past sequence, and treats the data without regard to chronology, the problem of economical storage requirements thereby is made impossible of solution, due to the fact that storage requirements are a function of sequence. As an illustration, a period of 10 years with wet and dry years alternating would require only a small reservoir for regulation, whereas the same flows, arranged in a sequence so that all wet years come first, followed by all dry years, would require a much larger reservoir for regulation; yet, in both cases, these data would give the same result when analyzed by duration-curve methods.

In this analysis, a hydrograph was constructed for the next 20 years based on certain requirements of probability and sequence derived from past records. This probable future hydrograph was then analyzed by mass diagram methods to determine economical storage requirements.

As has been stated elsewhere,[3] it is often convenient to plot the duration curve with the ordinate expressed as a percentage of the mean flow rather than absolute values. In most cases, the scale is selected so that the percentages of mean flow are equal divisions on the vertical scale. In this case, to avoid the computations necessary in reducing each flow to a percentage of the mean flow, the plot is made with absolute quantities, and the percentage of mean flows determined for those particular points desired in the study. The mean flow having been computed to be 536,000 acre-ft per year, Fig. 3 then shows that for 97.5% of the time the flow equaled 130,000 acre-ft per year, or 0.24 of the mean flow.

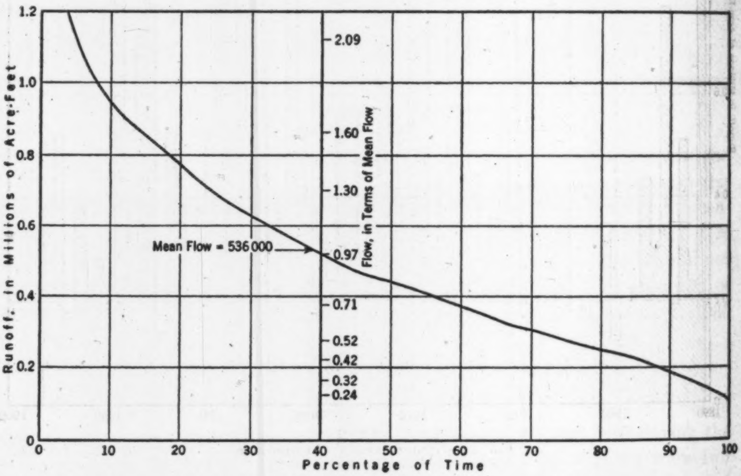

FIG. 3.—DURATION CURVE (CURVE APPLIES TO THE GAGING STATION AT CHRYSOTILE, ARIZ.)

With the flow established as a percentage of the mean flow, it becomes possible to compute the probable monthly flows, in terms of the mean monthly flows, for the various percentages of time.

The values of the probable monthly and yearly total runoff for various percentages of time are shown in Table 1. For the 20-year period the runoffs, in acre-feet, were:

Total probable	10,338,000
Average observed	10,800,000

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[3] *Transactions*, Am. Soc. C. E., Vol. 99 (1934), p. 1220.

The close agreement is not surprising, as the duration curve was integrated from the frequency curve, and differentiation of the duration curve must of necessity give the frequency curve.

TABLE 1.—PROBABLE RUNOFF (AT CHRYSOTILE), IN THOUSANDS OF ACRE-FEET

Month	Mean flow	TYPE OF YEAR								
		Flood	Very wet	Wet	Normal	Below normal	Dry	Very dry	Drought	Extreme drought
Percentage of time*	....	5	15	25	40	60	75	85	92.5	97.5
Flow in terms of mean flow......	1.00	2.09	1.60	1.30	0.97	0.71	0.52	0.42	0.32	0.24
Occurrences in twenty years....	....	2	2	2	4	4	2	2	1	1
January..........	46.7	97.6	74.7	60.7	45.3	33.3	24.3	19.6	15.0	11.2
February..........	74.8	156.5	119.6	97.2	72.6	53.1	38.9	31.4	24.0	17.9
March..........	105.0	219.5	168.0	136.5	101.8	74.6	54.6	44.1	33.6	25.2
April..........	95.2	199.2	152.0	124.0	92.4	67.6	49.5	40.0	30.5	22.9
May..........	48.1	100.5	76.9	62.5	46.6	34.2	25.0	20.2	15.4	11.5
June..........	18.7	39.1	29.9	24.3	18.1	13.3	9.7	7.9	6.0	4.5
July..........	21.1	44.1	33.7	27.4	20.5	14.5	11.0	8.9	6.8	5.1
August..........	31.2	65.2	49.9	40.5	30.6	22.2	16.2	13.1	10.0	7.7
September..........	24.5	51.2	39.2	31.8	23.8	17.4	12.7	10.3	7.9	5.9
October..........	17.8	37.2	28.4	23.1	17.8	12.6	9.3	7.4	5.7	4.3
November..........	24.2	50.6	38.7	31.4	23.5	17.2	12.6	10.2	7.7	5.8
December..........	32.9	68.8	52.8	42.8	31.9	22.9	17.1	13.8	10.5	7.9
Total..........	536.0	1,129.5	863.8	702.2	524.4	382.9	280.9	226.9	173.1	129.9

* Percentage of time in which a given flood type was exceeded.

TYPES OF YEARS

It is generally noted that all of the twelve months in a year follow the year type with respect to volume of water. In a wet year, of greater than average runoff, each of the individual months will be above average runoff for the month; and for the dry years, each individual month will be relatively dry. Thus, it follows, in general, that a wet year is composed of wet months, or, the infrequently occurring floods of a January most probably will be followed by the equally infrequently occurring floods of February.

For this study an arbitrary classification of year types was made: The infrequent "flood" year was exceeded only 5% of the time; the "very wet" year was exceeded 15% of the time; the "wet" year was exceeded 25% of the time; the "normal" year was exceeded 40% of the time; and so on, as shown by the headings of Table 1.

Again referring to Table 1, if the percentage of time is taken as a corresponding ratio to a given number of years (20 years, for example) the number of occurrences for each year type during the 20-year period can be obtained. If the runoff for each of the 20 years thus obtained were noted on a separate slip of paper, with the monthly values and total for the year, and the slips were drawn at random and noted in sequence, a hydrograph would be constructed which could be reduced to identical probability and duration curves of the observed runoff.

The number of different sequences which might be obtained by such a method is enormous,[4] but in every case the hydrograph so constructed would reduce to probability and duration curves identical to those obtained from observed data. Two or three trials will indicate the great range of sequence which may be obtained, and will show clearly that a mass diagram obtained from two different sequences may give widely different results, in spite of the fact that the hydrograph so constructed will meet conditions imposed, of similarity in probability to the observed runoff.

Although at present it does not seem possible to determine which of the many sequences will be the one that will be observed in the future, there are other data which will limit the number and establish certain conditions, thereby reducing the number of sequences possible and defining a limit of error. One limiting condition is imposed by the probability of prolonged drought. Although the driest year of record is 1900, with a runoff of 119,700 acre-ft, the driest 2-consecutive-year period is not twice the driest year, but a value slightly greater. A duration curve of 2-consecutive-year periods is shown in Fig. 4, from which it may be seen that an additional limiting factor is added to the probable future hydrograph: The minimum flow for any 2-consecutive-year period must equal or exceed 400,000 acre-ft.

Applied to the data deduced for the probable future hydrograph, this requires that the "drought" and "extreme drought" types of years be separated in the sequence, because the sum of these 2 years, if occurring in consecutive sequence, would be less than 400,000 acre-ft.

Although this condition of separation for two particular years will not reduce the large number of possible sequences very much, it does reduce the possible error by a large amount, because the critical periods of any project depending on river flow are the drought years. Similarly, the 3-year sequences are found to equal or exceed 700,000 acre-ft, imposing another limitation on the sequences of the probable future hydrograph.

In addition to the foregoing items, which may be used to limit the number of possible sequences in the probable future hydrograph, there is apparently a large field open for investigation in the realm of direct prediction of future flows.

FUTURE WEATHER

In 1932, the writer analyzed[5] some of the long-term trends in future weather, based on extrapolation of cycles observed in long-term records obtained from other sources. Since that time, and, perhaps, primarily due to the necessity of accurate knowledge of weather conditions as applied to aeronautics, considerable research has been done on weather trends and predictions. As is well known, commercial airlines are now able to predict weather a few days in advance with a high degree of accuracy.

Since 1932 George F. McEwen, of the Scripps Institution of Oceanography, has furnished interested parties with predictions for 1 year in the future, with a remarkable agreement between predicted and observed weather.

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[4] *Transactions*, Am. Soc. C. E., Vol. 91 (1927), p. 626.

[5] "Weather Records Projected into the Future," by John Girand, *Civil Engineering*, February, 1932, p. 76.

Although most projects are bonded for 20-year or 30-year periods, the first 5 or 10 are the most important, as success or failure is usually determined within that time. If, then, the trend of the weather for the next 10 years could be obtained with a reasonable degree of accuracy, and if the analysis would also show the worst condition which might occur in any length of time, the economic feasibility of the project could be determined. Accordingly, the probable future hydrograph should be constructed with particular emphasis on the next 10 years, and on the worst possible years.

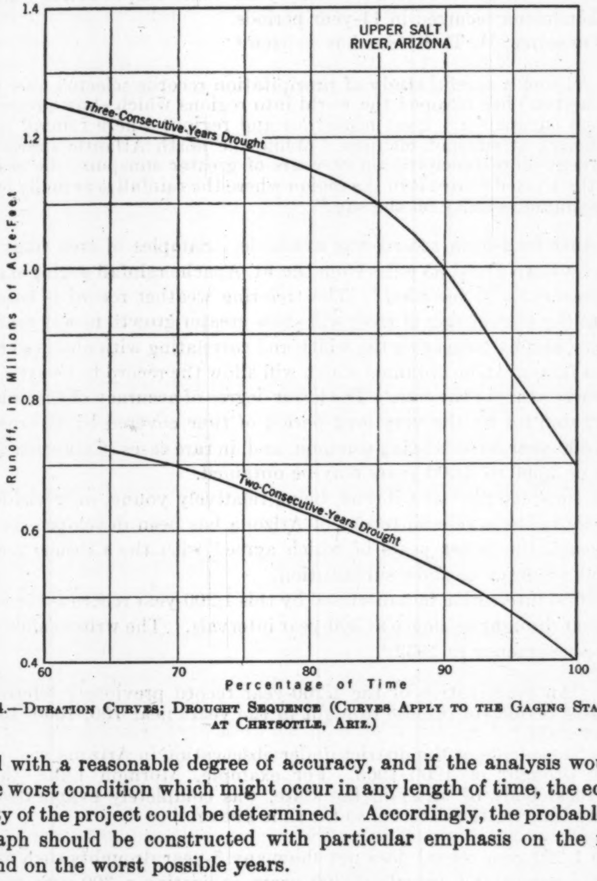

FIG. 4.—DURATION CURVES; DROUGHT SEQUENCE (CURVES APPLY TO THE GAGING STATION AT CHRYSOTILE, ARIZ.)

LONG-RANGE FORECASTING

It was necessary for the writer to obtain data from other sources covering long periods of time to detect long-term variations in weather. The 12-year record of the Chrysotile gage, although extended to 40 years by correlation with other gages, was still too short to do more than indicate possible trends.

Fortunately, other long-term records were available. The observed rainfall observations at Ft. Apache, a town centrally located on the watershed, covered a period of 60 years. An analysis of this and other rainfall records discloses a "cycle" which alternates in approximately 11 years. The discovery of the 11-year cycle in weather records opened the field of solar activity with respect to weather, as it had been known for many years that sunspot activity and related solar phenomena recurred in 11-year periods.

For example, H. T. Stetson has written:[6]

> "From a careful study of precipitation records selected over the globe [Clayton] has mapped the world into regions which show greater rainfall when sunspots are most numerous and regions where rainfall is actually deficient at sunspot maxima. While the north Atlantic shows 10 to 20 percent more precipitation in years of greater sunspots, the eastern half of the United States is in the region where the rainfall is actually less during maximum activity on the sun."

Another long-term record was available. Samples of tree rings taken at Cibecue, a town about 35 miles from the Ft. Apache rainfall gaging station, had been measured and recorded.[7] The tree-ring weather record is based on the fact that the annual ring of trees will show greater growth in wet years than in dry years, and by measuring the width and correlating with observed rainfall or runoff, a factor can be obtained which will allow the record to be extended back to the early years of the tree. The lower degree of accuracy of such measures is compensated for by the very long period of time covered by the record, 400-year to 500-year records being common, and, in rare cases (California Sequoias), records of 2,000 to 3,000 years may be obtained.

The trees sampled at Cibecue, being relatively young, only yielded a 269-year record; but a general record of Arizona has been developed for the past 1,200 years, the latter years of which agreed with the Cibecue record with sufficient accuracy to allow substitution.

The first interesting fact disclosed by this 1,200-year record is the recurrence of a 5-year drought sequence at 300-year intervals. The writer called attention to this phenomenon in 1932:[8]

> "An examination of the 1,200-year record previously referred to discloses periods of intense drought in the years near 700, 1000, 1300, 1600, and 1900 * * *.
>
> "Residents of the district under observation in Arizona well remember the drought of 1900–1905. For example, Mormon Lake, the largest natural body of water in the state, was completely evaporated, for the only time in its existence, so far as is known."

The 1,200-year record does not show any 5-year droughts such as this one occurring except at intervals of 300 years, indicating a 300-year cycle of this particular drought. Accordingly, the 1900–1905 sequence was neglected in all computations in which the sequence of dry years was involved in the probable future hydrograph for the next 20 years.

The second interesting fact disclosed by the 1,200-year record is the change in general climate during the Middle Ages. Archeologists had long wondered why the Indian pueblos of the Painted Desert in Arizona had been abandoned, but it was not until the 1,200-year weather record was developed that a satisfactory answer could be given. The record shows, clearly, a general decline in precipitation over a long period of years, which forced the Indians to abandon an area which became unsuitable for agricultural pursuits.

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[6] "Sunspots and Their Effects," by H. T. Stetson, McGraw-Hill Book Co., Inc., New York, N. Y.

[7] "Climatic Cycles and Tree Growth," by A. E. Douglass, Carnegie Institution, Washington, D. C.

Such a general decline for the present time is not observed in the last century of the record, indicating that the project will not be subject to any conditions other than those recognized as normal for the past century. Should such a condition start in 1940, the past record would seem to indicate that it would be many years before the project would have to be abandoned.

The 11-year cycle in tree-ring growth and its paralleling with the 11-year sunspot activity cycle is well known. In the 1,200-year record the parallel follows closely, even to the period 1645–1715, when there were no sunspots.

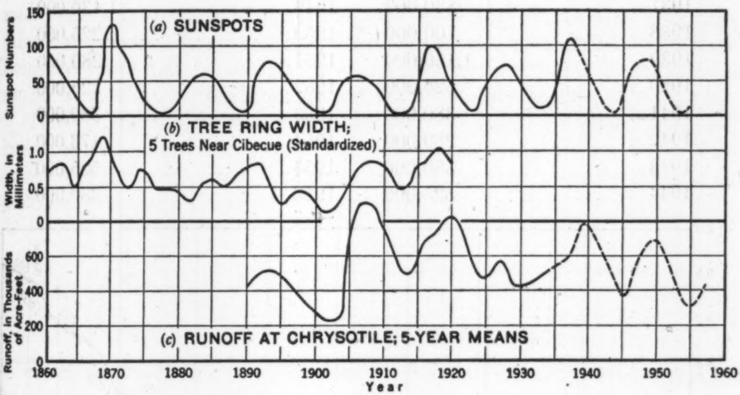

FIG. 5.—WEATHER TRENDS; UPPER SALT RIVER, ARIZONA

The definite cyclic trend of the sunspot cycle, as shown in Fig. 5, has been applied to tree rings, and correlated with the runoff on the Upper Salt River with such reasonable agreement as to allow a similar extrapolation to be made for the next 20 years. Although it is admitted that predictions of future sunspot maxima and minima are not always accurate, they do give a general trend of weather, which, when used in conjunction with the probable runoff as obtained from duration curves, will allow the construction of a probable future hydrograph which should represent conditions more closely than a sequence of flows drawn at random.

THE PROBABLE FUTURE HYDROGRAPH

From the data developed in the foregoing paragraphs, it is shown that the hydrograph of the future should meet the following conditions:

(*a*) Probability and duration curves derived from the future hydrograph must show agreement with the probability and duration curves derived from observed runoff;

(*b*) The arithmetic average of the 20-year period must agree with averages of observed records;

(*c*) The minimum 2-year and 3-year drought sequence must exceed 400,000 and 700,000 acre-ft respectively; and

(*d*) The general trend (a curve drawn through 5-year means) must follow the curve of sunspot activity. With these conditions in mind, the probable runoff, in acre-feet, at Chrysotile, was estimated as follows (the first three values being actual observations):

Year	Probable runoff, in acre-feet	Year	Probable runoff, in acre-feet
1933	370,000	1945	129,000
1934	178,000	1946	860,000
1935	653,000	1947	700,000
1936	580,000	1948	525,000
1937	830,000	1949	1,130,000
1938	700,000	1950	225,000
1939	1,130,000	1951	280,000
1940	525,000	1952	225,000
1941	380,000	1953	380,000
1942	280,000	1954	173,000
1943	380,000	1955	525,000
1944	525,000	1956	380,000

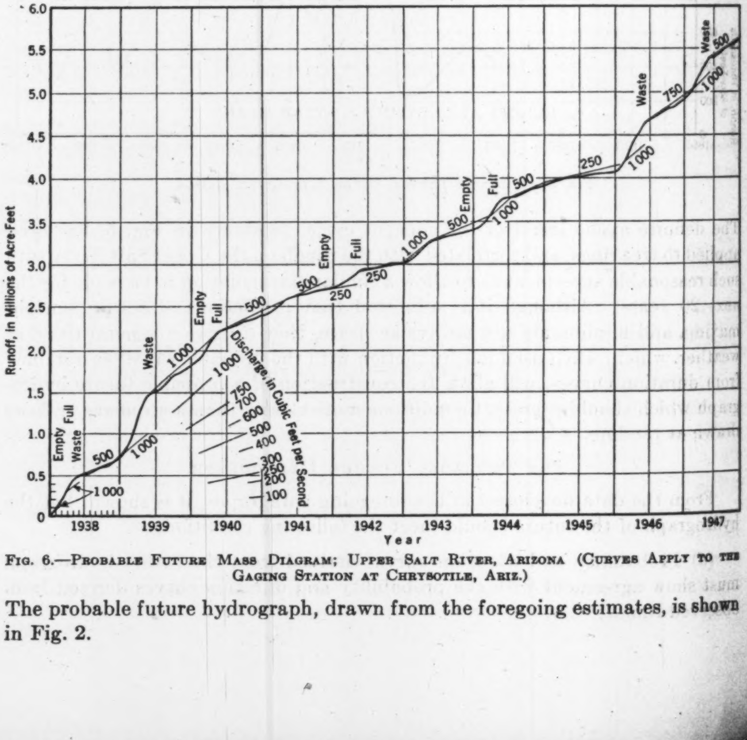

FIG. 6.—PROBABLE FUTURE MASS DIAGRAM; UPPER SALT RIVER, ARIZONA (CURVES APPLY TO THE GAGING STATION AT CHRYSOTILE, ARIZ.)

The probable future hydrograph, drawn from the foregoing estimates, is shown in Fig. 2.

It is admitted that other sequences of the years could be made which would meet the foregoing conditions; however, it is believed that such other sequences would not show great differences in actual operation of the project, bearing in mind that the large floods are lost over the spillways, and the periods of drought are the most important in the operation of the project.

From the probable future hydrograph, the economical storage requirements and power output were estimated by the mass diagram method. Fig 6 shows the conditions which obtained for a reservoir of 100,000 acre-ft.

CONCLUSIONS

The basic idea adopted for this analysis—that future flows are not entirely fortuitous—should not be a concept that is foreign to engineers when practically every other phenomenon met by the profession is a result of a definite cause. River runoff is a result of rainfall, which, in turn, is a result of evaporation by solar radiation. Although, admittedly, the knowledge of the laws governing rainfall and runoff is now limited, and the fact that rainfall and runoff data have been forced into skew probability curves indicate fortuity, fundamentally, there should be a reason for the sequence of flows which are observed.

Thus far, a combination of flows obtained by probability methods, arranged in such sequence as to satisfy what little is known of cyclic phenomena, may be expected to give a closer approximation to the truth than an analysis based on probability methods alone.

DISCUSSION

Dana M. Wood,[8] **M. Am. Soc. C. E.** (by letter).—Long-range predictions of rainfall and runoff have been attempted by many investigators, but so far with few practicable results. This does not mean that attempts should be discouraged. Sooner or later someone may find a method of sorting the many complicated actions and reactions of atmospheric conditions with their causes and effects and develop at least a hazy picture of what may happen during the next few years. At present, one can scarcely claim that predictions for the next few days or weeks are reliable.

The need for such predictions in power estimating is not great. (This does not apply to needed predictions for a few days for operating a power system.) Occasionally, it would be helpful to know whether the next year or two might be of wet, dry, or average type, in order to provide protection for some inadequacy during a development or construction period. However, careful analysis of sufficiently long and reliable runoff records, their proper interpretation, adequate and proper design of the power scheme, and the development of suitable operating guides and rules for obtaining the maximum efficiency for the purposes to be served are to be preferred to wild guessing as to future runoff. Good power planning takes into account the sequence of good and bad runoff years in the past on the theory that history may repeat itself. Provision should be made for the variations that surely will come by applying suitable contingency factors to the records. The length and general accuracy of the record determine to what extent it should be discounted. Storage, pondage, steam relay, and interconnections are all factors in the solution of the problem of insuring against the variations in nature.

Although careful consideration of Figs. 2 and 3 clears up any question in the reader's mind, it would nevertheless be helpful if they were designated as curves of annual total runoff to differentiate from the more usual monthly, weekly, or daily hydrographs and duration curves.

Table 1 is of doubtful value to any one. It is not correct mathematically; also, it assumes the same relative distribution during the year, regardless of its type. That this is far from what actually happens scarcely needs proof. This one assumption can lead to more error in estimating the potential power than even poor analysis and interpretation of actual records. The use of monthly estimates of runoff will ordinarily result in power estimates that are at least 10% too high, unless available storage is so large as to compensate for variations in flow during the month.

Many who have studied the problem believe that rainfall and runoff occur in cycles, varying both in amplitude and magnitude, just as, on the ocean, variations from the big wave to the little ripple depend upon whether the forces creating them are large or small, or whether they are acting mostly in one direction or opposing one another.

For purposes of making reliable power estimates, the writer advocates the analysis of runoff records, rather than rainfall records, because of the many

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[8] Prin. Hydr. Engr., TVA, Knoxville, Tenn.

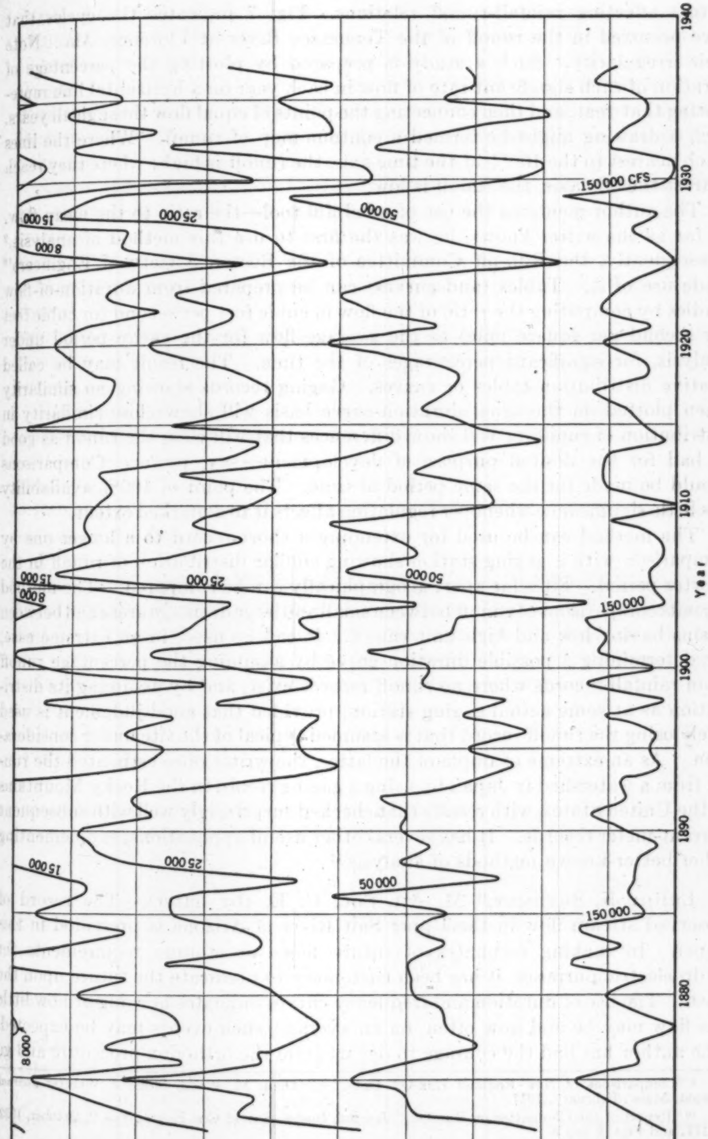

FIG. 7.—STUDY OF RUNOFF CYCLES BY CALENDAR YEARS, TENNESSEE RIVER AT FLORENCE, ALA.

factors affecting rainfall-runoff relations. Fig. 7 indicates the cycles that have occurred in the runoff of the Tennessee River at Florence, Ala. Note their irregularity. Such a study is prepared by plotting the percentage of duration of each significant rate of flow in each year on a horizontal line representing that year, and then connecting the points of equal flow through all years. Such a drawing might be termed a contour map of runoff. Where the lines reach nearest to the 100% of the time axis, the runoff is high; where they reach nearest the 0% axis, the runoff is low.

The author mentions the use of a helpful tool—the ratio to the mean flow. So far as the writer knows, he was the first to use this method of analysis.[9] Subsequently, the Run-off Committee of the Boston Society of Engineers[10] made use of it. Tables (and curves) can be prepared from duration-of-flow studies by computing the ratio of the flow in cubic feet per second (or cubic feet per second per square mile) to the average flow for the entire period under analysis, for significant percentages of the time. The result may be called relative distribution tables or curves. Gaging records showing no similarity when plotted on the usual duration-curve basis will show close similarity in distribution of runoff or will show differences that will class the runoff as good or bad for the desired purpose of development—say, power. Comparisons should be made for the same period of time. The point of 100% availability has little significance whenever regulation affects it to a marked extent.

The method can be used for extending a short record to a longer one by comparison with a gaging station showing similar distribution of runoff in the shorter period. Sites far apart geographically can be compared. The method permits comparisons of runoff between small and large drainage areas and between basins having low and high unit runoff. It can be used, in an extreme case, for determining a possible duration curve by assuming the percentage runoff from rainfall records where no runoff records exist, and by assuming its distribution as at some actual gaging station, provided that good judgment is used in choosing the runoff record that is assumed typical of the site under consideration. As an extreme example of the latter, the writer once estimated the runoff from a watershed in Japan by using a gaging record in the Rocky Mountains in the United States, with results that checked surprizingly well with subsequent current-meter records. It has several other useful applications, supplementing other better-known methods of analysis.[11]

LEROY K. SHERMAN,[12] M. AM. SOC. C. E. (by letter).—The record of observed stream flow in the Upper Salt River of Arizona is presented in this paper. In making estimates of future flows or storage requirements for hydroelectric purposes, it has been customary to predicate the future upon the past. The use of duration and frequency curves indicates how big or how little the flow may be and how often, on an average, such events may be expected. The author has had the courage to depart from the orthodox procedure and go into the future for at least 10 years. He makes specific prophecy when the flow events will occur. He compiles the mass diagram based in part on his forecasted 10-year hydrograph. Admittedly, a definite knowledge of flow conditions for the first 10 years of the life of a water-power plant would be valuable. The questions arise: Is the author's procedure safe for the requirements involved? What is the difference in amounts between the orthodox and the author's estimates? Does this affect the design?

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[9] "Comparisons of New England Run-Off Data," by Dana M. Wood, Stone & Webster *Journal*, Boston, Mass., February, 1917.

[10] "Report of the Committee on Run-Off," *Journal*, Boston Soc. of Civ. Engrs., Vol. 9, October, 1922, p. 171, and Figs. 2 and 3.

[11] Discussion by Dana M. Wood of "Rainfall and Run-Off Studies," by the late C. E. Grunsky, Past-President, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 106.

[12] Cons. Engr., Chicago, Ill.

Under the heading "The Probable Future Hydrograph" the author lists five control conditions. Conditions (*a*) and (*b*) are in accord with usual practice. Condition (*c*), "The minimum 2-year and 3-year drought sequence must exceed 400,000 and 700,000 acre-ft respectively," does not appear logical to the writer, despite the author's statement under the heading "Types of Years." Why should not these values be as low as, or lower than, the flow record for the 3 years 1902, 1903, and 1904—namely, less than 200,000 acre-ft? Condition (*d*), "The general trend * * * must follow the curve of sunspot activity," is a major feature of this paper.

It has been facetiously said that the sunspot theory shows remarkable correlation with precipitation after both events have happened. The writer has noted statements of one effect of sunspots in a part of the country and simultaneously the opposite effect in another part of the country. He is unable to reconcile this influence of the sun upon the earth, which is a little sphere rotating 93,000,000 miles away, at the rate of 1,000 miles per hr.

Nevertheless, the engineer should seriously consider any facts that seem to furnish some vision as to the definite or probable time of occurrence of meteorological events. He is expected to be a prophet. He will be a better prophet when he can say "the event will come within five years" instead of "the event may come some time during the next two hundred years—maybe next month."

In 1935, Sydney M. Wood correlated sunspots with corresponding elevations of the surface of Lake Michigan.[13] On this basis he forecast the elevations of the lake until 1949. A comparison with Mr. Wood's forecast and the actual elevations is as follows:

Date	Forecast in 1935	Recorded elevations, U. S. Lake Survey
January, 1936	578.2	577.91
July, 1936	579.4	578.65
January, 1937	578.7	577.78
July, 1937	580.1	578.66
January, 1938	579.0	577.74
July, 1938	579.7	579.55
January, 1939	578.8	578.74
July, 1939	579.8	579.96
November, 1939	579.3	579.35

Forecasts, as good as this, would permit the successful operation of regulating works on the Great Lakes.

With a few examples, like the foregoing, engineers will begin to have some confidence in sunspot forecasting. In the meantime, the theory should be applied with caution and the results compared with the more common procedures. If the forecasting should be in error, it is important to know how much the effect may be on either design or operation. In this connection, it would be an interesting comparison if the author had given the results derived from a mass diagram based only upon the observed hydrograph.

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[13] *Bulletin*, Associated State Eng. Societies, Chicago, Ill., October, 1936, pp. 83–102; see Fig. 1, p. 87.

At some time, in the course of events, there will be a 3-year average flow of less than 200,000 acre-ft per year. It has happened in the past. There is no gainsaying that it may not be even less than this. Likewise, the spillway will probably be called upon to carry a greater flood than any contained in the local record of gagings. The Division of Rivers and Floods of the U. S. Weather Bureau has been successful in deriving, for spillway design, a fair value for the maximum storm to be expected upon a basin. May not the sunspot theory be applied for a derivation of the maximum drought period?

It is fair to point out that Mr. Girand has proceeded with caution in delving into the future. In his forecasted 20-year period, his low flow for a year is 129,000 acre-ft versus 119,700 of record, and, for 3 years, 245,000 acre-ft versus 190,000 of record. The relatively small differences seem to be supported by the sunspot forecast. The author has performed a constructive service to the profession in presenting this specific example of forecasting stream flow on the Salt River.

GEORGE F. MCEWEN,[14] ESQ. (by letter).—A penetrating analysis of available information on stream-gage records and other correlated observations in obtaining the probable future 20-year hydrograph of the Upper Salt River, Arizona, is presented in the paper. Since an inspection of the 46-year hydrograph does not show a tendency even to approximate repetitions of sequences, the author concluded that extrapolation from this short series could not be made. However, he does use past information as a guide for the future by assuming that the observed frequency distribution of nine types of years—flood, very wet, etc.—will prevail in the future. Also, some restrictions as to possible future sequences are provided by preparing duration curves of two consecutive years, and again of three consecutive years, and thus establishing a probable minimum flow in the future for such intervals.

Skilfully, the author takes advantage of certain supplementary data and conditions in this arid region that are especially favorable to long-range forecasting. Although tree-ring measurements are indicators of the amount of precipitation, various other factors often have a great influence, thus obscuring any relation of tree-ring thickness to precipitation. However, in this arid Arizona region enough evidence has been obtained in support of a sufficiently close correlation to make it possible to interpret tree-ring measurements and to determine general trends and cycles in precipitation over hundreds or even thousands of years. From the correlation between precipitation and stream flow the author thus establishes certain important features of the Salt River flow. In particular, severe 5-year droughts such as that of 1900–1905 are thus shown to have occurred in 300-year intervals, beginning with the year 700 A.D. Therefore, the 1900–1905 sequence was neglected in all computations involving the sequence of dry years in estimating probable future sequences of dry years. Mr. Girand has followed the well-established conclusion that runoff data are not entirely fortuitous, and has accordingly realized that their investigation should not be based entirely upon probability methods. In fact, statisticians have long recognized that time series cannot be dealt with by the ordinary statistical methods because the observations ordered in time are in general not mutually independent or random in time. Although the author found no evidence of repetitions of shorter 2-year and 3-year sequences his 5-year means do indicate repetitions—a conclusion that is strengthened by correlations with the sunspot cycles which can be projected more readily. A note[15] confirms the author's 1944 year of increase and 1948 year of sunspot maximum.

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[14] Prof. of Physical and Dynamical Oceanography, The Scripps Institution of Oceanography, La Jolla, Calif.

In general, it appears that by the skilful use of available information, the author has succeeded in making a forecast of runoff for the 20-year period 1936 to 1956 that should well serve to answer questions as to whether operating requirements of a proposed hydroelectric project would be met.

JOHN GIRAND,[16] ASSOC. M. AM. SOC. C. E. (by letter).—The need for knowledge concerning future runoff probabilities in power estimating is questioned by Mr. Wood, who states: "Provision should be made for the variations that surely will come by applying suitable contingency factors * * *." Just what constitutes a "suitable" factor, or how such a suitable factor is obtained, is not stated. Although a "suitable" factor might be obtained on the Tennessee River, with 70 years of gaging records, the use of such a factor on the Upper Salt River with only 15 years of record might result in wide differences of opinions among engineers.

Fig. 7, the contour map of runoff of the Tennessee River, undoubtedly is useful in its place, but cyclic analysis requires smoothing of the hydrograph over long periods of time, rather than breaking up the hydrograph into magnitude parts for short periods of time. Under such conditions a contour map of runoff could scarcely be expected to be anything except irregular.

Mr. Sherman "leavens the loaf" with the statement that "* * * the sunspot theory shows remarkable correlation with precipitation after both events have happened." Such criticism might have been devastating had it not been followed by a quotation from the paper[13] of Sydney M. Wood giving forecasts of the elevation of the surface of Lake Michigan, based on the sunspot theory, and Mr. Sherman's admission that "With a few examples, like the foregoing, engineers will begin to have some confidence in sunspot forecasting."

For comparison, it is interesting to note that the results obtained by the methods outlined in the paper were about 10% lower than results obtained by orthodox methods; this constitutes an additional safety factor, and is in line with operating experience in this area.

Professor McEwen has presented the crux of the matter in his statement, "* * * statisticians have long recognized that time series cannot be dealt with by the ordinary statistical methods because the observations ordered in time are in general not mutually independent or random in time." This fundamental fact has been neglected by many engineers who still consider day-to-day hydrograph data as independent figures, any group of which may be selected for analysis without regard to considerations of whether the group represents average conditions.

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[15] "Forecast Next Sunspot Increase to Begin in 1944, with Maximum in 1948," by J. Q. Stewart and F. C. Eggleston, *Bulletin*, Am. Meteorological Soc., Vol. 21, March, 1940, p. 117.

[16] Asst. Hydr. Engr., James B. Girand, Phoenix, Ariz.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 2107

CHANNELIZATION OF MOTOR TRAFFIC

By Guy Kelcey,[1] M. Am. Soc. C. E.

With Discussion by Messrs. W. L. Waters, C. J. Tilden, T. M. Matson, W. W. Crosby, Julian Montgomery, R. M. Reindollar, Irving Mack, Bruce D. Greenshields, T. W. Forbes, James S. Bixby, George H. Herrold, Hawley S. Simpson, Arthur G. Straetz, Burton W. Marsh, Virden A. Rittgers, and Guy Kelcey.

Synopsis

The motor car provides great individual freedom, range, and flexibility of movement. Many millions of these motor vehicles are operated, at high speeds, along a multiplicity of intersecting roadways. Although most drivers, both amateur and professional, have acquired surprising skill, traffic accidents and congestion have become major public problems.

Channelization, which refers to means by which certain aspects of this problem may be attacked, proceeds on the principle that traffic safety and efficient roadway use are each necessary to, and functions of, the other and that both are products of order. By confining movements of vehicles to definite lanes, channelization avoids or reduces disorders arising from conflicting movements. In part, too, it accomplishes its purposes by simplifications to which drivers respond naturally. It utilizes the skill and ability of drivers as a whole and, to a considerable extent, prevents or neutralizes faulty driving. The result is a self-regulated flow which insures orderly operation of vehicles by drivers as individuals and by groups.

Channelization

The history of civilization is a record of transport which, in its turn, is a story of channels. Making use of many kinds of transport for himself and his possessions, man traveled and "took root," through the centuries, mainly over water and along natural land channels, which followed the most convenient courses between major, distant points. Caravan routes, the Santa Fe and Oregon trails, Mississippi River traffic, and even modern air routes are examples.

Note.—Published in December, 1939, *Proceedings*.

[1] Signal Service Corporation, Elizabeth, N. J.

Man settled the narrow strips of good land adjacent to these routes of travel. At major junctions, bulges of population developed into the important cities of today. These over-crowded peaks of population are a consequence of earlier forms of transport, which provided conveyance between major points but did not do a good job of local distribution. The motor car, which is distributing these bulges of population more uniformly and over wide areas, provides greater use of existing channels, is creating a multiplicity of new ones, and is reaching others not hitherto accessible. It has multiplied, by many times, man's speeds, his range, and the number of courses along which he may travel and adjacent to which he may settle.

All important transport, even on wide oceans and in the three-dimensional air, operates, then, within the limits of quite definite and very narrow paths. As these paths become crowded, or because they are not exactly fixed, those who use them must conform to quite exact rules. In particular, these rules are made to keep travel within the limits of directional or one-way channels so that opposing and cross movements shall be in least interference. Any failure to adhere to these rules and to keep within assigned channel limits reduces flow efficiency and increases hazard. The establishment and strict application of such rules has not proceeded as far with respect to the operation of the motor car as it has with relation to other forms of transport.

As a newcomer in the transport system, the motor car has, in a sense, shouldered its way into the general scheme of things. It is to be expected that its intrusion into, and its impact on, existing institutions, practices, and habits should have many dislocating and jarring effects. On the other hand, it is the only means of power transport available to the non-professional and individual operator generally, and these amateur drivers constitute the great majority of automobile operators. It is amazing how competent many of them are, what a slight degree of control they actually need, and what a complete and easily controlled device the motor car has become. In fact a study of the accident experience involving a large representative group of drivers in Connecticut (particularly accident repeaters), for a period of six years, discloses that only 1.3% of all drivers are accident repeaters and that 80.9% are normally efficient.[2]

The surprising fact, then, is not the congestion, the accidents and other problems which the motor vehicle has introduced, but that so much progress has been made in so short a time in the evolution of such a vast, new and vital means of transport without the development of problems that are much worse.

There are so many automobiles, and they move so fast that it has not been possible, by police or other regulation, to force all drivers to conform exactly to the rules that are aimed to insure orderly movement. Neither has it been possible to educate or force each driver to keep in his own place or channel and to keep out of wrong ones because most roadways permit and often invite wrong movements. It is the writer's contention that if proper directional channels are made logical, easy, and inviting to use, and if others are made unattractive and difficult, drivers (who, in the main, are surprisingly competent anyway) will follow lines of least resistance and use the right ones. Thus quite a number of the traffic problems that bother traffic officials will cease to exist for lack of cause. The subject of this paper—channelization—refers, then, to the means by which good driving is made easy and bad driving difficult.

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[2] "The Normal Automobile Driver as a Traffic Problem," by T. W. Forbes (of the Bureau of Street Traffic Research, Yale University, New Haven, Conn.), *Journal of General Psychology*, 1939, pp. 471–474.

Of course, the conception of one side of the road being used for cars traveling in one direction and the other side for cars going the other, even if the road is not physically divided, is elemental. This rule "Keep to the Right" (or left, in some countries) is so thoroughly comprehended by all persons that it constitutes a foundation upon which the solution of any traffic problem must be based.

It is interesting to report that Mr. William Phelps Eno[3] defined and adapted many of the principles of channelization early in this century by applying them to the solution of problems introduced by increasing concentrations of carriages, bicycles, and early motor cars in New York, N. Y., Washington, D. C., and in Europe. The creed of the Eno Foundation for Highway Traffic Control, "Ex chao ordo" (out of chaos, order) is a wholly satisfactory introduction to the writer's subject, and to the statement that efficient, smooth-flowing, safe, motor traffic is a product and consequence of order. Channelization refers to means by which traffic may be made self-regulating and orderly.

The motorist is often charged with errors which, in reality, are natural consequences of disorder arising from inadequate or faulty roadway conditions that invite such errors. Fluid traffic flow and safety, which are functions each of the other, are both products of order; and order, in turn, is one function of proper roadway design.

Channelization, which is concerned primarily with problems of the pavement surface and above the surface, establishes proper lanes within which movements of vehicles are confined. It increases the utility and safety of automobile transport by simplifying traffic flow and by eliminating confusions, conflicts, and eddy currents. It takes advantage of the high degree of skill and instinctive driving capacity possessed by a vast majority of drivers and does much to neutralize the unsatisfactory performance of a few drivers who may not drive properly, either occasionally or habitually. Finally, it invites and encourages good driving manners, bad ones being quite often more an evidence of confusion or desperation than of poor sportsmanship.

HUMAN FACTORS

In seeking causes of traffic difficulties and in applying corrections, several factors, the more important of which are mentioned herewith, must be kept in mind.

Instinct Factors.—Many driving practices, including most good ones, naturally become habits, which, in turn, become instinctive. Timing and coordination, which are instinctive factors, are as necessary to good driving as to good music. In working with traffic, it is important to establish conditions favorable to the natural exercise of these instinctive qualities so that the driver's mind may be on the alert, and free to "captain" the vehicle, to navigate, and to attend to small adjustments and driving emergencies.

Course of Least Resistance, Speed, and Simplicity.—A driver tends to take the most convenient course, the course which calls for the least effort, causes least discomfort, and requires least reduction of speed. Channelization, which blocks wrong courses or makes them difficult, makes correct ones easy, convenient, and inviting.

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[3] "Street Traffic Regulation," 1909, p. 7, Sections 437 and 440, pp. 10, 11, 28, 29, and "The Science of Highway Traffic Regulation," 1920, pp. 26–39, Eno Foundation for Highway Traffic Control, Saugatuck, Conn.

Traffic solutions should be based on the fact that drivers tend to operate at speeds up to the limit of their feeling of security and comfort, but few drivers press beyond this limit. Although this speed factor varies, depending on the driver, the car, the road, the weather, and other conditions, approximate speeds at which normally good drivers (and this means most of the traffic load) will operate, under a given set of circumstances, usually can be predicted or arranged in advance.

Advertisers tell it to the "Zilches" and the "Stuyvesants" understand. Channelization, which is designed to be understood by the few who are poor drivers, offers no difficulties to the great majority who are good ones.

Heads Up.—When boys walk railroad rails those who keep their heads up go farthest. A structural worker on "high steel" fixes his attention ahead. To walk a line one must look at a point that is up and ahead.

A good driver must keep his head and his attention up. He does not, then, so much approach, as pass, objects 100, 200, 300 or more feet in front (depending on speed and circumstances). Thus, while his attention comprehends both near and far range of roadway in front, his major interest is focused well ahead, observing and evolving his plan of action and checking it constantly. Except in the case of necessity, he gives near ranges checking and glance attention only. The more open the road and the greater his speed, the farther ahead is his attention focused.

Roadway design should, and channelization treatments must, be laid out with this factor in mind. What the driver is to do must unfold and be sufficiently apparent from a distance so that his major attention may be given to other matters on closer approach, and so that he may continue to operate his vehicle with his head up. Since channelization eliminates irregular vehicular movements, a driver's attention is not distracted by the treatment itself or by the possibility of surprises from other vehicles. Therefore, he is free to form, and to execute, good judgments with respect to the fewer and lesser problems that remain, both immediately in front and far ahead of him.

Since channelization takes advantage of instinctive driving propensities, the success of a channelizing treatment will usually be in proportion to its glance or instinct value. Such a treatment must have no surprises or "ambushes" and must be so well thought out and designed that it need not be solved consciously by the driver. A driver goes in only one direction at a time and, progressively, should make only one decision at a time. If the treatment is well done, he is never in doubt because his course and his next action will always have a one-step-at-a-time simplicity.

DRIVING FACTORS

Merging, Intersecting, and Opposing Movements.—In the illustrations that follow, arrows indicate courses and directions of vehicles. However, arrows cannot represent, accurately, the effect of a moving vehicle as a zone of influence that varies with velocity and is subject, as well, to driver characteristics and other factors. Thus, a standing car influences only its area. When this car moves, it commands leeway on both sides and clearance ahead which, at speed, is many car lengths in extent. The intersection of the courses of vehicles, then, involves moving zones of influence with the actual vehicle approximately centered, as a core, in the rear of each zone.

When two cars operate in the same direction, the movement is a merging one and the relative speed of the merger approaches the difference of their speeds, as a limit. Thus cars *A* and *B* (Fig. 1), moving parallel to each other at 45 miles per hr and 40 miles per hr, have a relative speed of 5 miles per hr.

When two cars, *A* and *E* (Fig. 2), intersect at a right angle, each maneuvers with respect to the other at his own speed. Each driver works to a definite reference point and, subject to visibility and high-speed factors, each gages the actions of the other with reasonable accuracy because, relatively, each moves across, rather than along, the other's range of vision.

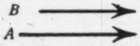

FIG. 1

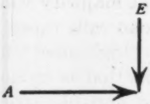

FIG. 2

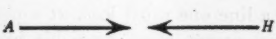

FIG. 3

Two cars, *A* and *H* (Fig. 3), moving in opposite directions at 45 miles per hr and 40 miles per hr, have a relative speed which is the sum of their speeds, or 85 miles per hr. The exercise of good judgment in the face of high, relative, head-on, speed is often difficult, where the courses of vehicles intersect, because each driver moves along (as distinguished from across) the other's range of vision and neither has any definite reference point to guide to.

The movement of two cars in the same direction and parallel to each other is most efficient and the safest. Progressively (see Fig. 4), as the movement of cars becomes increasingly opposed (as *A* and *B*, *A* and *C*, *A* and *D*, *A* and *E*, *A* and *F*, *A* and *G*, and *A* and *H*) conditions favorable to natural or instinctive driving decline. Uncertainty and hazard increase and efficiency of movement is reduced. The increasing relative speed of car *A*, at 45 miles per hr, with

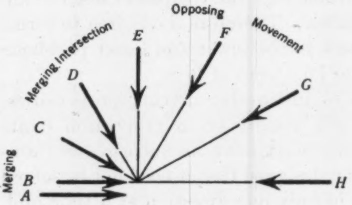

FIG. 4.—SPEEDS OF *B*, *C*, *D*, *E*, *F*, *G*, AND *H* MOVING AT 40 MILES PER HR RELATIVE TO *A* AT 45 MILES PER HR

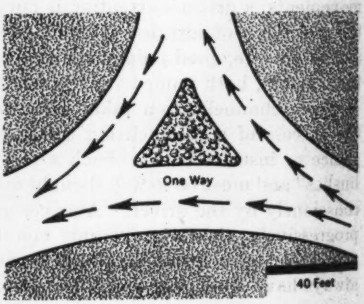

FIG. 5

respect to *B*, *C*, *D*, *E*, *F*, *G*, and *H*, at 40 miles per hr, is indicated by scaled lines in Fig. 4. This increase in relative speed ranges from the difference of speeds, as in the case of *A* and *B*, to the sum of the speeds, in the case of *A* and *H*.

As traffic movements become increasingly opposed, confusion and uncertainties develop and multiply. Under more complicated circumstances, involving fixed objects and many moving vehicles from various directions, in combination with a large number of possible paths, quite complex situations of rapidly changing relationships often upset the natural rhythm of driver actions. Resulting disorders are progressive and, once started, are most difficult to control. Channelization accomplishes the easier objective of preventing the introduction of disorders.

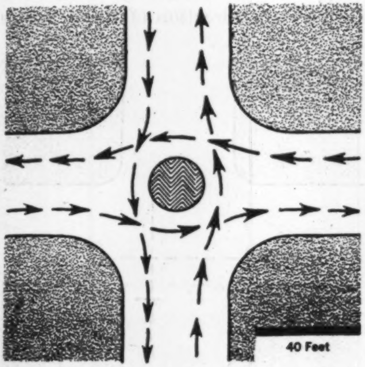

FIG. 6

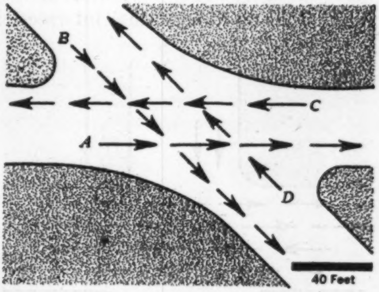

FIG. 7

A traffic circle (Fig. 5), with a diameter of several hundred feet, would provide a merging movement of vehicles which would be essentially parallel. Traffic flow would be continuous, and maneuvering of vehicles with respect to each other would be at the difference of their speeds.

If the diameter of such a circle is reduced gradually, the courses of vehicles will merge and finally cross at steeper and steeper angles which approach a right-angle intersection (as in Fig. 6) as a limit.

To go beyond a right-angle intersection creates opposing movements which should be avoided. In Fig. 7, path *A* merges with path *B* but opposes path *D*. Path *C* merges with path *D* but opposes path *B*. Thus, the value of two merging movements is more than offset by two opposing movements.

From the foregoing, conclusions may be drawn that, where roadways intersect, the merging movement created by rotary traffic is safest and most efficient and that these advantages gradually reduce as the radius of the rotary movement decreases. If a rotary movement cannot be established so that all movements merge, then a merging intersection or a right-angle intersection (which a rotary movement approaches as a limit) is to be preferred and designs that create opposing movements are best avoided.

Intersection Efficiency.—When continuity of flow (which occurs when intersecting roadways enter a traffic circle so that vehicles merge in a rotary movement) cannot be arranged, certain inefficiencies, which result from interrupted or shuttling movements, must be kept in mind.

The maximum flow capacity of an intersection of two roadways at grade cannot exceed the capacity of the roadway with the greatest number of feeder lanes. Since the number of feeder lanes of important intersecting roadways is usually equal, the maximum possible capacity of a normally important intersection cannot exceed half the capacity of all approach or feeder lanes—that is, of the same intersection with roadways separated (see Fig. 8)—since, if one road keeps full, the traffic on the other has no chance to cross. This 50% maximum efficiency can then be attained only when traffic is flowing continuously along one roadway, without interruption. The following discussion of flow efficiency at a normal intersection, where the direction of traffic flow is alternated constantly, is then in terms of some portion of this maximum possible, 50% efficiency, and is based further on the assumption that traffic is feeding in from each direction as fast as the intersection can absorb and clear it.

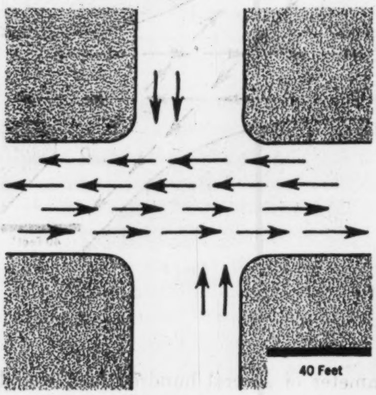

Fig. 8

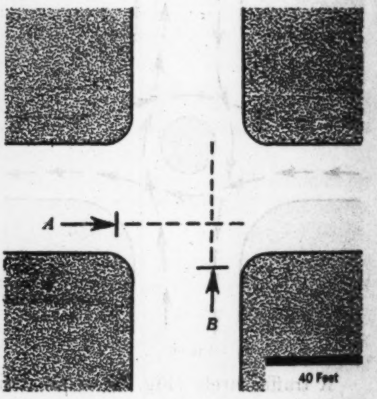

Fig. 9

Assume a simple, right-angle intersection (Fig. 9) equipped, for purposes of this example, with stop-and-go signals. When the red signal stops traffic *A*, a 3-sec or longer intermediate clearance interval is allowed normally for that traffic to clear the intersection, after which traffic *B* gets the green "go" signal. These 3 sec are lost because the main movement, during this time, is at a standstill in both directions. If the cycle of signal operation is 60 sec, this clearance interval time represents 5% dead loss per reversal. Since there are two reversals of traffic movement per cycle, a total loss of twice 5%, or 10%, occurs, per signal cycle, at the usual simple right-angle intersection, in clearing the intersection so that alternate crossing movements may be released.

When traffic *B* gets the green signal, cars in line do not spring into instant motion. Unless he "jumps" the amber light, the first driver will normally lag in starting from a half to a full second, after the green signal appears. The second car will lag, in starting, behind the first and so on. Thus, under heavy load conditions, a favorable green signal will complete its interval and change again to red by the time a line of cars has "accordianed" out and the end cars are started. Therefore, if a flow of vehicles is stopped and then restarted a considerable period elapses before efficient lane flow is again attained. Principal reasons are driver perception and reaction lag in starting, acceleration lag, a confusion or eddy current lag, and, at intersections, delays arising from turning movements abnormally impeded by the "bunching" of cars.

It will be obvious from the foregoing that deceleration and clearance losses in stopping a flow of vehicles in one direction, and losses in getting started by the crossing movement, introduce a very definite inefficiency every time traffic movements are reversed. These losses would not occur (as an example) if the crossing roadways were separated.

Consider now any intersection which is not at a right angle, as Fig. 10. Traffic *C*, with much farther to go to clear the intersection, takes 5 sec to clear. This adds to 10 sec lost in a two-interval cycle or $16\frac{2}{3}\%$ loss, if a 60-sec cycle is used. Traffic *D* is as inefficient in starting as before and now has farther to go to refill the intersection, thus putting it back into full use. Therefore, intersections at other than right angles are even less efficient, when considered in terms of alternating movements, than intersections where roadways cross at right angles.

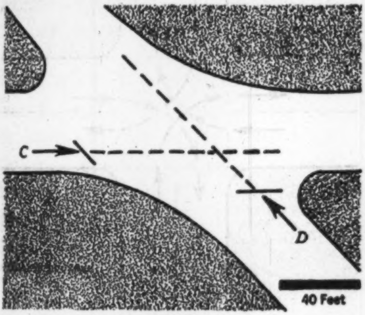

FIG. 10

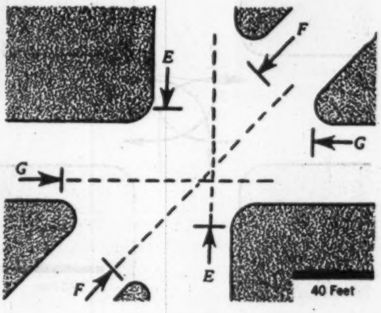

FIG. 11

At complicated intersections, as in Fig. 11, where three-interval cycles are used, these losses are often greater. Traffic *G* requires at least 5 sec to clear the intersection when the green light disappears. Traffic *E* is subject to the same losses, as shown for Fig. 10, in restarting and in again attaining reasonable flow efficiency. These losses occur in each reversal. Thus, in a three-interval cycle at this intersection of, say, 75 sec total length, 15 sec or 20% may easily be lost per cycle in stopping and clearing each movement to make way for one of the other crossing movements. If the losses, occurring when a line of vehicles restarts, are added to the stopping and clearing losses it will be appreciated that a two-interval signal cycle is to be preferred to a three-interval cycle.

An objective of channelization is to create conditions favorable to the most efficient traffic flow at each location by simplifying movements of intersecting vehicles, by reducing the range of alternation or amplitude of shuttling, by channelizing to reduce to a minimum, the number of reversals per cycle where signals are used, and, if a continuous merging movement cannot be effected, by channelizing traffic flow to insure that vehicles intersect at or near a right angle. Thus, in the case of the multiple intersection shown by Fig. 11, channelization can be used often to establish merging movements or to reduce the range of alternation or shuttling quite substantially and, if signals are necessary at this intersection, to reduce the usual three movements to two per signal cycle.

Left Turn.—Theoretically, all cars turning left should circle the center of an intersection. Practically, another type of left turn is usually more desirable. As shown in Fig. 12, cars turn left around the center at such short radius that even those with short wheel bases find the turn difficult, whereas a truck, or a car with a long wheel base, often finds it impossible to make the turn without interfering with other lanes of traffic. It will be noted that cars turning left, from each direction, describe paths that intersect at eight points. With this type of turn, intersection "jams" or "lock ups" occur frequently.

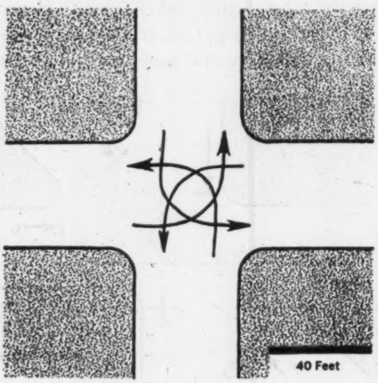

Fig. 12

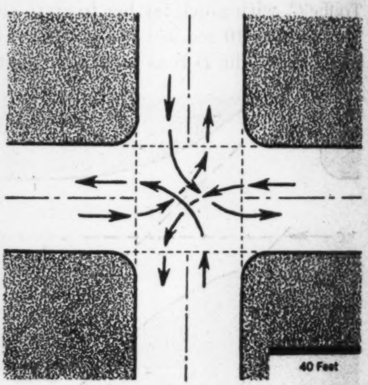

Fig. 13

Another, and much more natural left turn, is one in which the north-bound car, Fig. 13, turning left (west), enters the intersection from a position to the right of the street center line and leaves to the right of the center line of the street on the left. All four cars, each turning left, describe paths which intersect at only four points. This "inside left turn" is efficient, fast, safe, and natural.

Channelization creates conditions at intersections which permit the "inside left turn" but make its abuse difficult. Some of these abuses are shown in the following examples.

Approach from Behind and Eddy Currents.—A traffic accident is not a premeditated act but occurs when something unexpected and unlooked for happens. One course of least resistance that results in unexpected traffic movements, disorder, and accidents at intersections is the short-cut, left turn.

Obeying a natural impulse, a driver follows the most convenient, straight-line or long-radius course in an unrestricted intersection or traffic area. If (as is usually the case) the course followed is a wrong one, he goes too fast, operates in and across wrong lanes and comes unexpectedly upon other cars from head-on, sum-of-the-speed angles.

Driver *A*, Fig. 14, cuts a short corner. Taking a long-radius course, his speed is high. He gets into two wrong channels. Like a man walking diagonally across a street intersection, he must now look in three directions at once to watch out for the possibility of cars *B*, *C*, and *D*. Driver *B*'s attention at the moment is properly devoted to driver *C*. Driver *C* gives his attention first in the direction of driver *D* and then in the direction of driver *B*. None of them is looking for, or expecting, the movement of driver *A*. While car *A* merges with car *D*, it intersects the courses of cars *B* and *C* at opposing, sum-of-the-speeds angles. Pedestrians *W*, *X*, *Y*, and *Z* are watching cars *B* and *C*. They do not expect, or look for, car *A*. In each case driver *A* comes upon other drivers and pedestrians unexpectedly and from behind their respective ranges of vision. Drivers *B*, *C*, and *D* and pedestrians *W*, *X*, *Y*, and *Z* abandon natural or instinctive driving and walking habits and do unexpected things that now make them dangerous to others and to themselves and which are characteristic of the disorderly and hazardous, traffic eddy current. Such eddy currents are progressive and tend to increase from minor to major complication.

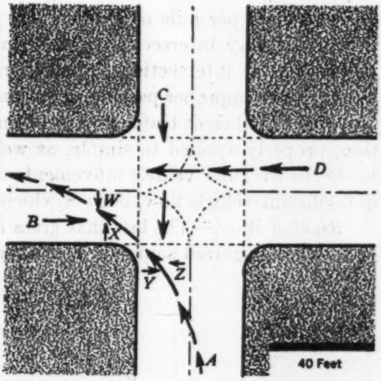

FIG. 14

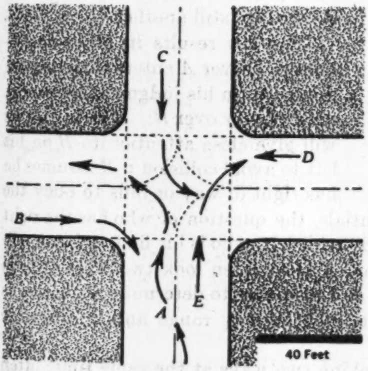

FIG. 15

Traffic eddy currents are illustrated by the sequence of events leading to an actual accident shown in Fig. 15. Driver *A*, making a long-radius, short-cut turn at speed, passed driver *E*. Because visibility was obscured by a building, driver *A* did not, at first, see driver *B*. With right of way over driver *C* and looking in the direction of driver *C*, driver *B* started across the intersection. Driver *A*, to avoid driver *B*, deflected right forcing driver *C* to the left, and driver

E, dodging driver *C*, crashed into car *D*. This eddy current movement in which one car, cutting short and from behind, forced other cars progressively into unexpected opposition at sum-of-the-speed angles, is a common occurrence which may be observed in some degree at any ordinary intersection, if the movement tracks of cars are traced, during a period of light traffic, on an intersection diagram.

In round numbers, 75% of traffic accidents in cities and 25% on highways occur at intersections. Of course, there are many more intersections per mile of street than per mile of road so this percentage difference does not indicate that a highway intersection is less dangerous than one in town. Irregular movements at intersections, arising principally out of high-speed, short cuts and turns, or improper passing, are a major cause of these intersection accidents and of the inefficient traffic flow so often conspicuously apparent. Channelization, properly applied to simple, as well as to complicated, intersections, prevents the irregular vehicle movements that are a cause at once of accidents and of inefficient vehicle flow, both of which result from the same disorderly causes.

Right of Way.—The law that gives right of way to the driver on one's right is a little-recognized source of confusion where vehicles intersect.[4]

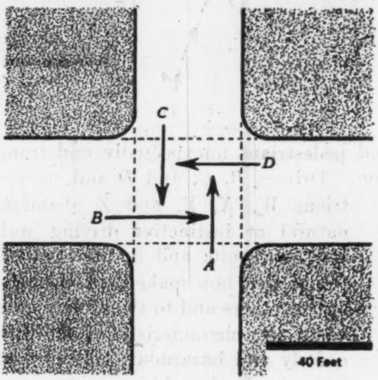

Fig. 16

Under the law that gives right of way to the driver on one's right (Fig. 16), driver *A* has right of way over driver *B*, driver *B* over driver *C*, driver *C* over driver *D*, and driver *D* over driver *A*. In the exercise of this right, a driver may find himself blocked in the middle of an intersection by another driver who is in turn blocked by still another.

The law results in other confusions. Driver *A* enters the intersection with, in his judgment, apparent right of way over *B*. If *A* is wise he will give close attention to *B* on his left to avoid collision if *B* assumes he has right of way or fails to obey the law or if, due to speed or other differentials, the question of who has the right of way is uncertain. Concerned with *B* on his left, who is his first and immediate problem on entering the intersection, *A* must often look two ways at once and settle a problem of relative speeds and distances to determine if *D*, who is on his right, is within an approximate right-of-way range and is therefore entitled to it or not.

In this problem (which involves looking two ways at the same time, often from a position within an intersection), swift appraisal of a number of varying factors, and quick decision, confront a driver each time his course intersects those of other vehicles. As a matter of fact, the writer is not familiar with any rule by which a driver can know exactly just when another driver on the right is entitled to right of way—whether it is when two cars are a block apart, 100 ft apart, or 50 ft. This makes the problem of determination, which is troublesome enough anyway, doubly difficult.

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[4] See "Simplification of Highway Traffic," by William Phelps Eno, 1929, pp. 15 and 16.

Assume that the law were changed to give right of way to the man on the left, as illustrated by Fig. 17. From a position outside of, rather than within, the intersection, driver *A* gives major attention to *B* who is now not only the one whose path he crosses next but also the driver to whom he must yield right of way if *B* is within or entering the intersection. This points to a suggestion of a definite and easily established reference point, namely: "if *B* is within or entering the intersection" he has the right of way over *A*. When *A* does enter the intersection his right-of-way relationship with *D* is similarly simplified for both drivers; that is, when *A* reaches the curb line he has right of way over *D* if *D* is outside; but he does not if *D* is entering or is inside.

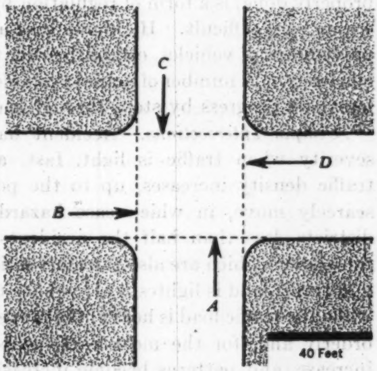

FIG. 17

Thus, if right of way were given to the driver on the left, center of the intersection, lock ups now resulting from the opposite rule would be avoided, and the problem of who has right of way would be simplified greatly. Problems arising from having to decide two things, and to look two ways at once—toward the man on one's left whose course one intersects next and the man on one's right who is entitled to right of way— would be reduced or avoided.

Furthermore, the question of who is to have the right of way next would be determined by drivers from a distance equal to the roadway width apart, before they enter the intersection, and therefore from a position of safety. Drivers would not have to make this decision as they are entering or are actually in the intersection exposed to collision or, in any case, in close range contact with other vehicles, as is the case under the present rule.

When cars travel on the left side of the road and the driver sits in the right side of the car as in England, the rule that gives right of way to a driver on the right is easily obeyed because it is instinctive. In the United States, where one drives to the right and sits in the left side of the car, to yield right of way to a driver on the left is instinctive and natural whereas giving right of way to a driver on the right is unnatural, less efficient, and unsafe.

Channelization cannot correct, but it may often be applied to modify, confusions arising from this reversed regulation at ordinary intersections. It reduces confusion caused by this regulation in large traffic areas. This is particularly true where, in a channelization treatment, merging movements of vehicles can be arranged.

Applications

The foregoing is an approach to necessary understandings and points of view of the engineer who studies a traffic situation with the object of creating a design of roadway that blends driving requirements and driver characteristics into a satisfactory solution. Such an engineer understands that channelizing, properly done, is a form of regulation by which right actions are made easy and wrong ones difficult. He knows that agreeable restraints, to prevent irregular movements of vehicles, establish order that avoids accidents and promotes flow efficiency. A number of actual examples follow. These start with simple problems and progress by steps through more complicated and difficult situations.

Simple Intersections.—Accident hazard is greatest as to likelihood and severity when traffic is light, fast, and unrestrained. Hazard declines as traffic density increases, up to the point of such dense traffic that vehicles scarcely move, in which case hazard approaches zero. In many business districts, less than half the accidents occur during business hours and the remainder (which are also the most serious) occur when the business day is over and traffic load is lightest. If vehicle track patterns in intersections are traced while the traffic load is heavy, the movement of cars will be observed to be quite orderly and, for the most part, proper. As traffic density declines, speeds increase, and patterns become increasingly irregular, as indicated by Fig. 18. Left turns particularly change from orderly movements to high speed, short cuts.

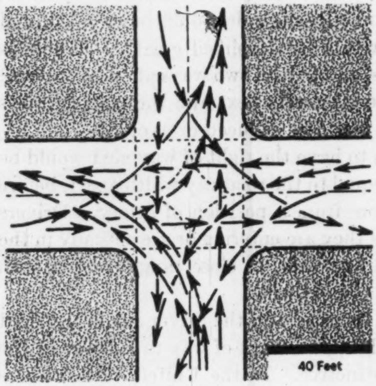

Fig. 18

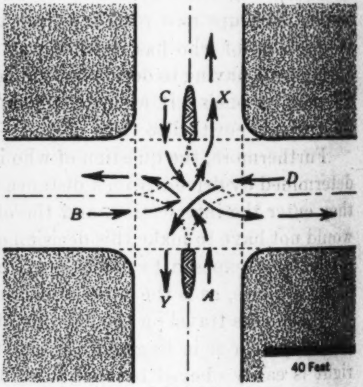

Fig. 19

One function of channelization consists in the treatment of ordinary intersections to correct these disorders. Thus, the most convenient course for car *A* in clearing the lower island, Fig. 19, is to drive straight ahead and to begin a left turn at or beyond the end of the island. Starting a turn from this position and making it at maximum comfortable speed (which drivers may be counted on to do), car *A* will enter the street to the left correctly, even if it is a wide one—that is, to the right of its center line.

Similarly, driver *D*, turning left, will clear the lower island most conveniently by continuing into the intersection to the right of the center line of the street he is leaving, thus making a natural turn and dropping straight in past the island. The upper island similarly regulates the movements of vehicles *B* and *C*. Thus two islands placed in opposite throats prevent irregular turning movements in the intersection.

To state this in another manner: Drivers *A*, *B*, *C*, and *D* find correct movements easy and natural and wrong ones not only awkward but actually difficult and inconvenient. Pedestrians are safeguarded by islands of refuge and by the fact that traffic movements are orderly and predictable.

If stop-and-go signals are used at this intersection they may often be installed, particularly in cities and towns, on pedestals on these islands where, in addition to their normal functions, they signalize the islands; and, from this position, they usually have best visibility to drivers and to pedestrians. Such treatment has the value, also, that the signal need be operated "stop" and "go" only during rush hours and the channelizing treatment meets requirements, at other times, with signals flashing amber, thus permitting continuous, safe movement of traffic.

Wide Throat Intersections.—Given the opportunity to roam over an enlarged area, drivers accept the invitation and invariably proceed along paths which are the shortest or fastest courses between any two points, as shown in Fig. 20. It will be noted that vehicles intersect at steep, head-on angles, approaching each other at nearly the sum of their speeds. The short-cut, vehicle paths and speeds "too great for conditions" on left turns are not supposition. If the opportunity is there, it will be taken advantage of, except during the periods of heavy load.

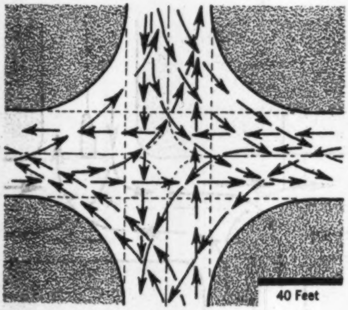

Fig. 20

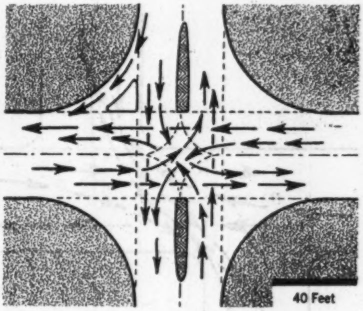

Fig. 21

The treatment used in the correction must be carried from the curb line back somewhat behind the point where the roadway is of normal width (see Fig. 21); otherwise the invitation to cut short remains. The short cut must be blocked.

Enlarged Throat, Dead Ends.—The short-cut, long-radius, high-speed movements indicated in Fig. 22, produce head-on, sum-of-the-speed, vehicle intersections. Pedestrians, particularly, have a difficult time in these situations because it is often not clear, at any point, from which direction the next car will come. Drivers, also, find themselves in the same predicament.

Two corrective treatments are indicated in Figs. 23 and 24. It will be noted that both treatments block wrong courses and make right ones easy and inviting.

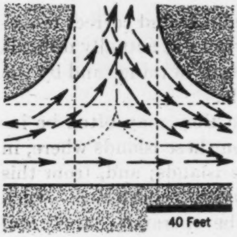

FIG. 22

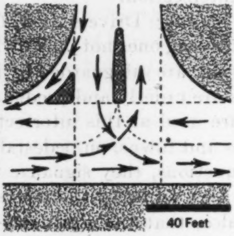

FIG. 23

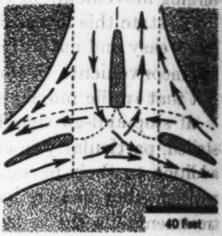

FIG. 24

Note, too, that, in all cases, intersection of vehicle paths is about at right angles and that turning movements are easy and normal. Pedestrian refuge islands on the sides as indicated on Fig. 23 may be desirable sometimes.

Offset Intersections.—The type of intersection shown in Fig. 25 usually develops the compound crisscross of movements indicated. Examination of

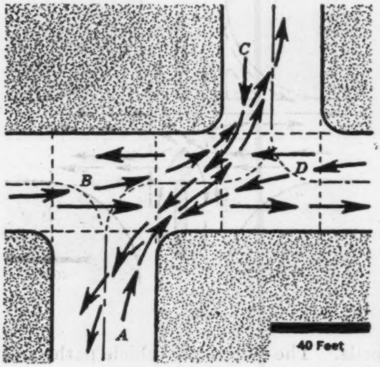

FIG. 25

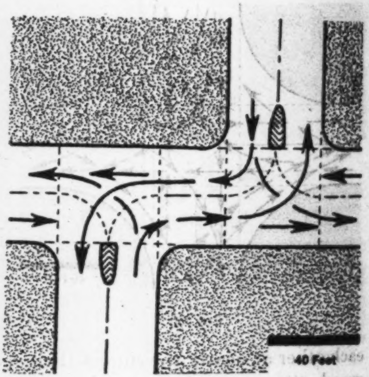

FIG. 26

curb faces at the lower right and upper left hand corners often discloses abrasion and tire marks from traffic *A* and *B* which strike at the upper left and traffic *C* and *D* which strike at the lower right. Wheel-mark patterns on the pavement usually confirm how completely offside drivers do get when the intersection layout invites such movements. The treatment shown in Fig. 26 corrects faulty movements and provides pedestrian safety in the two offset street throats as well.

Angled Roadway Intersections.—The angled street type of intersection invites a long, speedy short cut in which cars *A* and *B*, Fig. 27, approach head on. Because of the distance between curbs, the pedestrian hazard is greater than normal. If no left turns are made into the angled roadway the treatment indicated in Fig. 28 is effective. Where left turns are a factor, as in Fig. 29, the island may be reduced to permit a free, easy-radius, left turn. In cases of heavy pedestrian traffic, the right-hand island, as shown in Fig. 29, may be desirable. In any of these treatments, pedestrians, as well as drivers, have only one direction to look at a time. Traffic *A*, the conflicting element, without being unreasonably inconvenienced, is forced into a right-angle intersection, at a moderate speed, with traffic *B*, and is no longer a menace to pedestrians, to other drivers, and to itself.

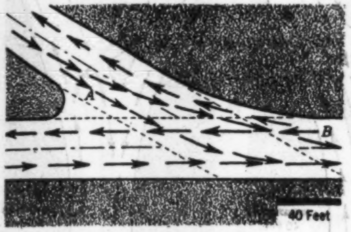

FIG. 27

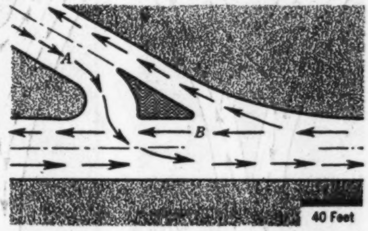

FIG. 28

Y-*Intersections.*—Although the construction of Y-intersections should be avoided, many exist to plague the driver and the traffic official (see Fig. 30). They invite the type of head-on intersections of vehicles at steep, sum-of-the-speed angles that are the cause of continuous disorder. Such conditions render judgment difficult and change instinctive and natural driving to the uncertain, unpredictable movements of a driver who proceeds on the basis of separate observations and of separately thought out actions. Such drivers, whose timing now goes askew and who do erratic things, are often mistakenly charged with carelessness.

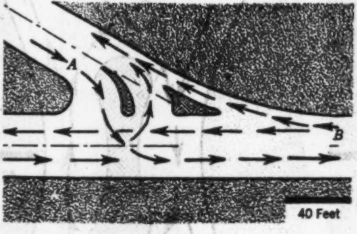

FIG. 29

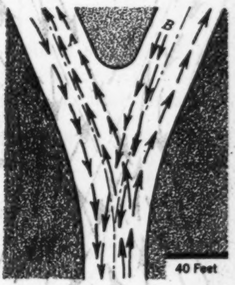

FIG. 30

Treatments shown in Figs. 31, 32, and 33 block off wrong courses. They cause traffic *A* and *B* to intersect at a right angle and at an observable, predictable point. Note that the axis of the treatment may be swung to accommodate an equally distributed load, or to give greater convenience to a heavy traffic load in one direction. Note, too, that islands used extend back far enough to prevent cutting short.

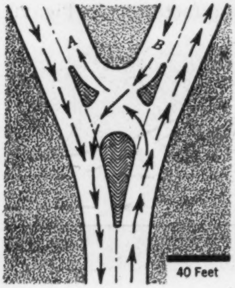

Fig. 31

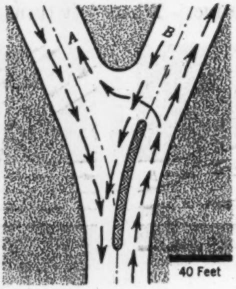

Fig. 32

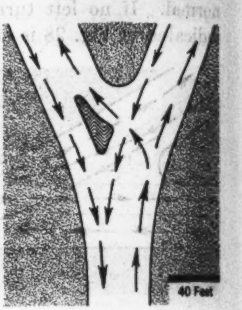

Fig. 33

X-*Intersections.*—The X-intersection is a double Y with two extra head-on movements added so that the problem is compounded (see Fig. 34). Drivers often find instinctive judgments incorrect because the actions of other drivers are unnatural and unpredictable. An eddy current involving one or two cars

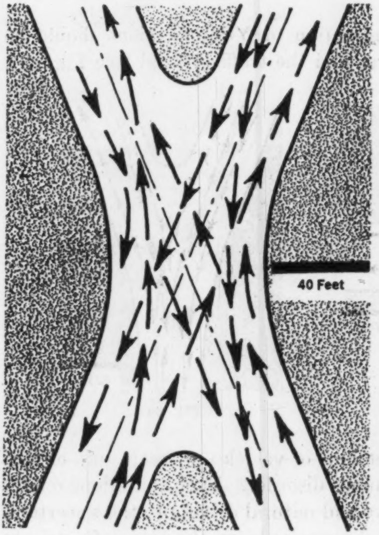

Fig. 34

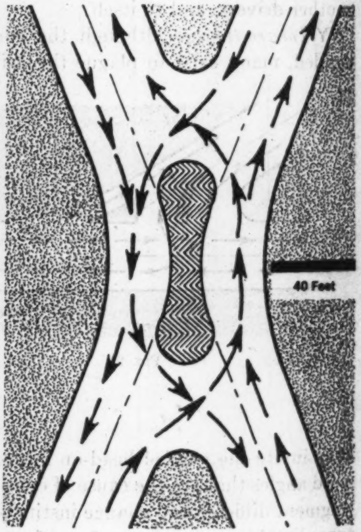

Fig. 35

suddenly spreads to a number, for quite a distance back in two or more directions; and chaos, which always threatens, often follows.

The axis of the correction is usually arranged to favor heaviest load, as indicated in Figs. 35,[5] 36, and 37. Definite reference and intersection points are established and orderly movements give instinctive driving abilities more chance.

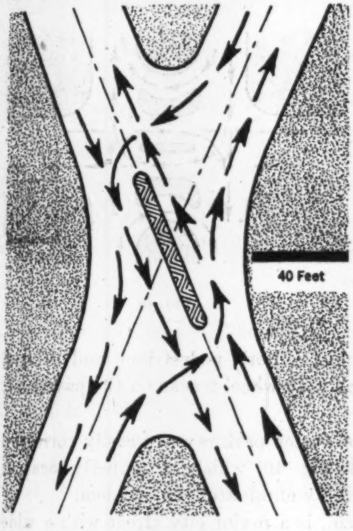

FIG. 36

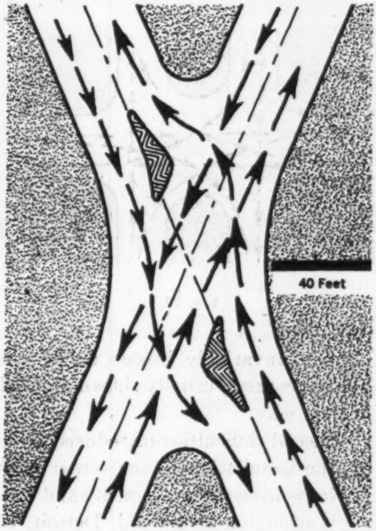

FIG. 37

Treatment to Favor Traffic Load.—A good example of swinging the axis of a traffic island treatment to favor the heavier load is to be found in the "Miracle Mile" near Phoenix, Ariz. (see Fig. 38). In this case, the main loading *A* and

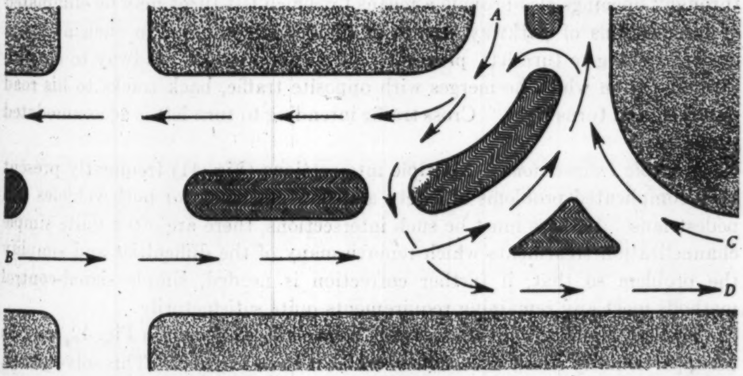

FIG. 38

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[5] See "Simplification of Highway Traffic," by William Phelps Eno, 1929, p. 74.

B is convenienced, whereas *C* and *D* are forced into slower paths which merge in the case of *C*, *B*, and *A* and, in the case of *B* and *D*, intersect on the merging side of a right angle.

Parkway Intersections.—The center parkway type of intersection, shown in

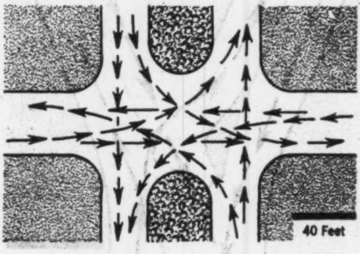

Fig. 39

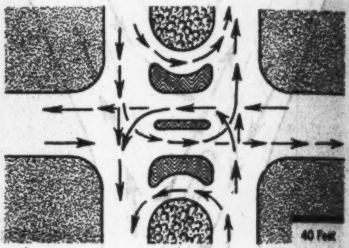

Fig. 40

Fig. 39, invariably breeds much trouble for both pedestrians and drivers. Vehicle courses, usually shown quite clearly by wheel tracks on the pavement, explain why.

Special difficulties introduced by the center parkway are easily corrected by the installation of the center island, Fig. 40, which, if properly located, prevents irregular movements and invariably eliminates the problem.

Washington Boulevard, Detroit, Mich., is a major city street with a wide parkway down the center. There is a heavy U-turning movement of traffic to hotels, offices, and shopping districts. "Turn through" openings are provided in the parkway as shown in Fig. 40. These openings may be near the intersections or closer to the center of the block, as conditions dictate. In either case, they permit a car to execute a U-turn without crossing pedestrian lanes or getting into interference with traffic in the intersections. These "turn through" openings also provide a means by which left turns may be eliminated at intersections of parkway streets or divided highways. In such a case a driver, wishing to turn left, proceeds beyond his intended roadway to a "turn through" from which he merges with opposite traffic, back tracks to his road or street, and turns right. Cross traffic intending to turn left is accommodated similarly.

Multiple Intersections.—Multiple intersections (Fig. 41) frequently present very complicated problems of safety and flow efficiency for both vehicles and pedestrians. If there must be such intersections, there are often quite simple channelization treatments which remove many of the difficulties and simplify the problem so that, if further correction is needed, simple signal-control methods meet any remaining requirements quite satisfactorily.

The half circle or built-up sidewalk treatment, as shown in Fig. 42, may be used, particularly where main flow of traffic is to be favored. This solves most, if not all, pedestrian and vehicular difficulties.

Quite often an original design, or a redesign of an existing situation, will permit the installation of a complete circle with deflection islands, as in Fig. 43. In such cases a very satisfactory solution is the result because resulting order prevents accidents and insures flow efficiency.

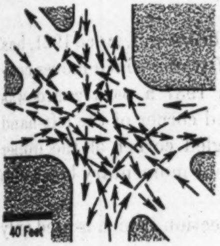

FIG. 41

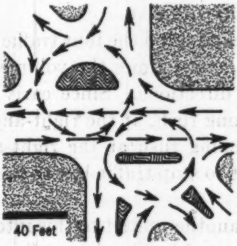

FIG. 42

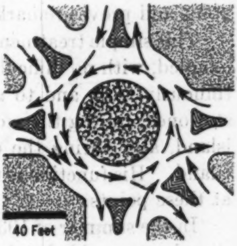

FIG. 43

Large Traffic Area.—Channelization of traffic areas is well illustrated by the problem shown in Figs. 44 and 45, which is an outstanding example of results to be accomplished by simplifying traffic flow and applying other principles referred to in this paper.

Prior to March, 1921, in the Lincoln Monument area of Fairmount Park, Philadelphia, Pa., three officers and a sergeant were used to handle traffic during rush hours. Each driver suited his own convenience as to routing. When the flow of cars was heavy, congestion and lock ups occurred; at other times speed became a factor. There were nine major points of intersection at which vehicles approached each other from head-on, sum-of-the-speeds angles. Since there were no definite channels for traffic, a driver could not be sure what others would do. Resulting disorder caused accidents, tangles, and confusion; and, during rush hours, it was the cause of serious congestion with long lines of blocked cars, particularly in the direction of the heavy flow.

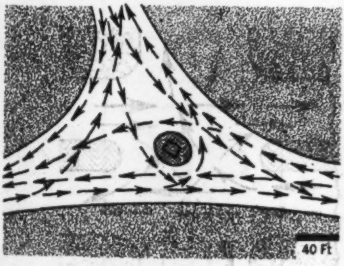

FIG. 44

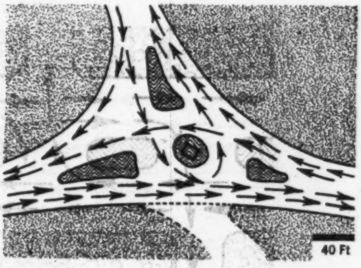

FIG. 45

The channelizing treatment, shown in Fig. 45, simplified traffic flow. It confined vehicles to definite channels, obstructed wrong movements; proper paths were made convenient, many intersection points were reduced to three

in which vehicles intersected at right angles, pedestrians were provided with safety zones, unexpected high-speed approaches from behind were eliminated, and drivers and pedestrians had the confidence of knowing what to expect and where to look. Note that the island on the left shades traffic turning left around the monument into East River Drive. This offset is needed at this point and proves remarkably effective.

This simple treatment eliminated the four traffic officers and, since 1921, has handled, with no difficulty, many times the volume of traffic which moves continuously from and to all directions. Since early in 1937 a park guard was stationed, during the morning rush, at the right-angled corner of the left-hand island, and during the evening rush at the right-angled corner of the upper island. His function is not to stop traffic but to prevent momentary blockades at these points.

In the summer of 1938 another outlet to this intersection, which is used only slightly, was added as indicated by the dotted line.

Divided Highways.—The divided highway is a channelizing device by which traffic flowing in opposite directions is separated by a physical barrier such as a center island or curb. The principle of separation of opposite flows of traffic is capable of enlargement to accomplish other functions including that of rotary movement of intersecting traffic. Since this type of treatment has been covered most ably by the late Fritz Malcher,[6] the writer's observations are limited to certain directly interesting and important aspects.

Several applications of the divided highway are encountered in practice. One method, illustrated in Fig. 46, provides openings in the center island for crossing into or out of every driveway and street. In this case, much of the advantage of the divided highway is lost because through drivers face the constant double threat that a car in one high-speed lane may slow down and pass through one of the many dividing spaces into or across the other high-speed lane in entering or leaving a local entrance. On a divided highway with many center openings, a moment's inattention may be serious because the entering

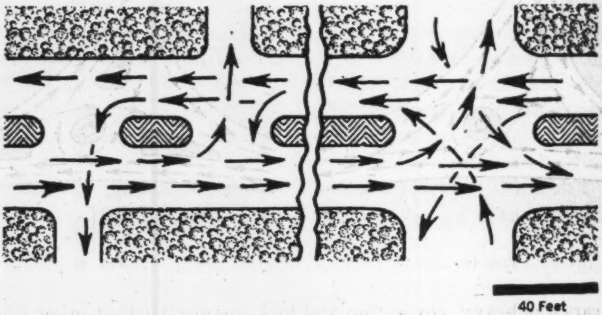

Fig. 46

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[6] "The Steady-Flow Traffic System," by Fritz Malcher, Harvard City Planning Studies, Vol. IX, 1935.

and leaving movements referred to, into and out of the high-speed lanes, are often wholly unexpected, confusing, and dangerous, particularly after dark.

Another method (shown in Fig. 47) considers that the convenience and safety of the heavy flow of traffic on the main highway are more important than the convenience of a few vehicles from the side, if they are not too greatly imposed upon, and provides openings through the center or barrier section only at intersections. This reduces the points at which main flow traffic need concern itself about crossing vehicles and concentrates cross flow so that "stop" and "go" signals, or other control measures, may be provided and justified.

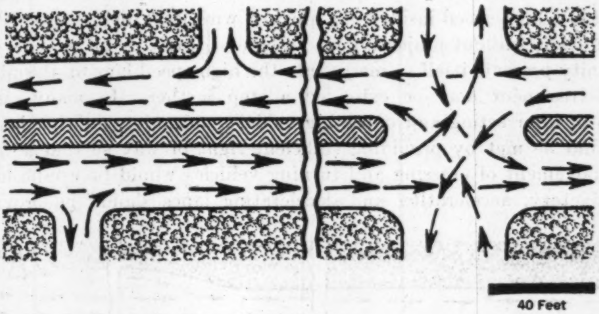


Fig. 47

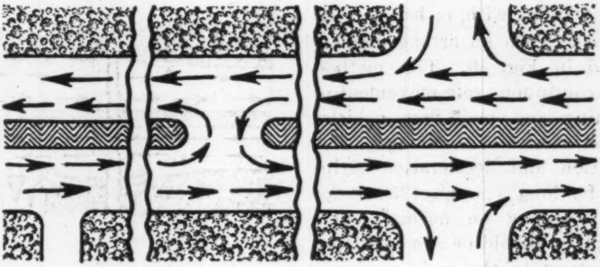
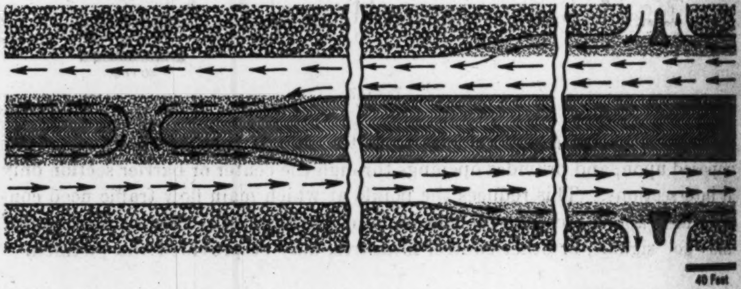


Fig. 48

A third method (see Fig. 48) provides openings through the center barrier at intervals often 1,000 to 1,500 ft or more apart. Such openings should be midway between, or at least some distance from, intersecting streets or roads. This establishes a form of rotary traffic which, if handled properly, results in merging movements of cross and through traffic. The greater safety, promptness, and certainty of safe departure from, entrance into, and crossing major highways, with this treatment, may be expected to find support from both local and through motorists, particularly on some main four-lane highways that carry such heavy loads that sometimes it is almost impossible to cross them without the aid of an officer or a "stop" and "go" signal.

One important factor in this treatment consists in a dividing island that is at least wide enough, or has barrier openings that are long enough, or has a combination of length and width, so that vehicles can cut into the center openings, out of a high-speed inside lane, quickly, with least disturbance. Vehicles can then wait, without projecting into a lane of traffic on either side, until an opportunity presents itself to merge into the high-speed lane on the other side.

This treatment may be extended a step further. In many cases, the problem of intersecting and departing vehicles on major, multiple-lane, highways could be met by providing sufficient right of way so that a very safe rotary movement of crossing and turning vehicles would be established. To be satisfactory, accelerating and decelerating lanes should be provided for vehicles merging with, or leaving, the main flow. Such an arrangement is indicated in Fig. 49. This method permits continuous safe movement of intersecting and main-flow vehicles with least interference or danger. The acceleration and deceleration strips should be long enough (350 ft or more, depending on normal traffic speeds); they should be of a color that is in contrast to the main roadway, and should preferably be unattractive to the high-speed, main movement and yet be inviting to traffic merging into, or departing from, the main highway.



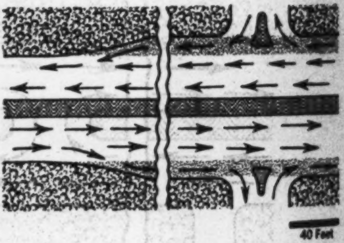


FIG. 49

Where, on main highways, openings through the center islands can be spaced sufficiently far apart, the center island may be tapered down to slight width, in center sections, as shown, thus reducing land use and overpass or bridge widths. The designers of the Merritt Parkway in Connecticut used this method of providing ample turning radius through island openings, of saving valuable land space between, and of reducing overpass and underpass widths.

Grade Separation.—At the intersection of major roadways, sometimes channelization may be used to postpone the need of providing costly grade separations (see Fig. 50). Since such need is usually anticipated well in advance and long before the intersection becomes a major problem, sufficient land may be acquired at such locations to permit the installation of an adequate traffic circle. When a separation finally does become necessary, the need for it will have been postponed, the land, originally secured at a lower cost, will have been utilized for rotary traffic in the interval, and a sufficient land area will have been provided for the separation and approaches.

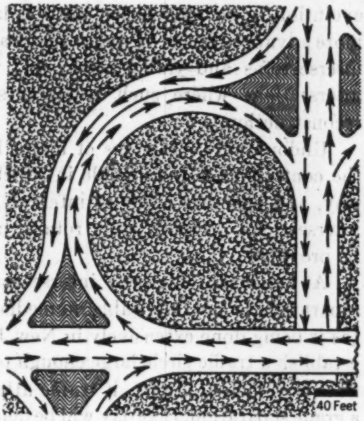


Fig. 50

The sketch, which is used in this instance as a symbol (Fig. 50), shows a quadrant section of a typical grade-separation of the clover-leaf type. Many variations of this treatment are used. All depend for their success on channelization principles. Channelization, therefore, is not only necessary to the success of these advanced practices in highway design but is an element in postponing or eliminating the need for such expensive treatments and in reducing their final cost.

Pedestrians

The problem of the pedestrian has not been placed last because it is considered unimportant, but for the reason that, if complicated vehicle movements are made orderly, the question of what to do with, or about, the pedestrian is solved—or at least simplified. The pedestrian was not born confused. Rather he is a creature of his environment, the victim of the high velocities and unexpected attacks to which modern traffic exposes him. These are the cause of his uncertain movements and hesitations.

Channelization removes the element of surprise from the pedestrian problem. Proper channelizing insures the pedestrian that he need look only in one direction at a time and that he will not be "ambushed" by vehicles that attack suddenly from behind his range of vision. Also he is provided with an island or refuge of ample width at intervals of one or two traffic lanes upon which he can orient himself, safely, before he crosses the next channel in which, also, vehicles will move in one direction only.

When the confusions, fears, surprises, "ambushes," and alarms introduced by vehicles moving swiftly from expected and unexpected directions are eliminated, the pedestrian is master of his own destiny to a surprising extent.

After establishing orderly traffic movements and pedestrian refuges by channelizing, if foot passengers are still in need of a special officer, signal, or other aid at any point, such aid will usually be necessary only to a limited degree and during certain hours rather than for long, or all-day, periods.

CONCLUSIONS

Beginning with the principles applied in handling bicycle traffic, horse-drawn vehicles, and early automobiles, there has been a slow but continuous evolution of channelization until today these principles are being applied in more advanced forms on major roadways, in the reorganization of many existing intersections and traffic areas, and in the solution of traffic difficulties at intersections where such treatments eliminate the need for redesign and reconstruction.

Undoubtedly, designers of major highways in the future will make use of the continuous flow principle in which a center barrier strip divides the highway, and openings occur at intervals, but not opposite or closely adjacent to intersections. Thus a truly rotary, merging movement of crossing traffic will be provided.

At important highway intersections it seems probable too that, if grade separation is not advisable immediately, sufficient ground will be acquired (as is being done extensively in New Jersey where a large number are in use) to establish a traffic circle large enough to provide a continuous, smoothly merging flow of vehicles. Later, if traffic attains volume and speed enough to require a grade separation, that day will be deferred by the efficiency of the rotary movement. The circle will provide the land, initially secured at lower cost, in the meantime serving a valuable purpose, upon which to place a grade separation.

By permitting continuous, orderly flow at reasonable speeds, channelization helps drivers to their destinations more quickly and safely than the dash-slow-down-dash-stop-and-dash procedure otherwise necessary.

A more general application of these principles will assure fluid movement of motor vehicles and will reduce, greatly, the confusions, and eddy currents that are the cause of accidents.

Plans for, and the administration of, motor transport of the future must include greater attention to driver characteristics and to his problems and needs—that is, to the problems of traffic from the pavement surface up. To accomplish these purposes, a greatly expanded knowledge and application of channelization will be necessary.

ACKNOWLEDGMENTS

In addition to names cited directly in the text, valuable suggestions, ideas, and criticisms were received from other engineers, traffic officials, and laymen almost too numerous to mention. At various times, officials of such state highway bodies as Arizona, Maryland, Minnesota, New York, Ohio, and Washington, and traffic experts of the cities of Philadelphia, Pa., Kansas City, Kans., and Chicago, Ill., and of the American Automobile Association, the National Paving Brick Association, and the American Transit Association, have contributed to the work. The writer acknowledges particularly, and with sincere appreciation, many valuable suggestions, ideas, and criticisms from the following: Miller McClintock and Maxwell Halsey, Yale Bureau for Street Traffic Research; Robert A. Mitchell, traffic engineer of Philadelphia; E. V. Miller, Assoc. M. Am. Soc. C. E., design engineer of Arizona Highway

Department; Lacey Murrow, Assoc. M. Am. Soc. C. E., director of highways of the State of Washington; Harry E. Neal, traffic engineer of the Ohio Highway Department; W. F. Rosenwald, traffic engineer, Minnesota Highway Department; the late George Schlesinger, M. Am. Soc. C. E., National Paving Brick Association; T. J. Seburn, traffic engineer of Kansas City; Leslie Sorenson, traffic engineer, City of Chicago; Peter J. Stupka, Works Progress Administration, Washington, D. C.; and many others.

These acknowledgments would be incomplete without mention of the writer's immediate associates of many years in the American Gas Accumulator Company and Signal Service Corporation, who helped to establish, refine, and define the principles of channelization described in this paper and to evolve the language by which its requirements have been expressed.

DISCUSSION

W. L. Waters,[7] M. Am. Soc. C. E. (by letter).—The channelization of vehicle traffic by separating islands at street crossings has probably been practiced for 100 years in the large cities of Europe; and, although Mr. Kelcey's paper is somewhat academic, it is most valuable as a collection of up-to-date ideas on the subject. As is frequently the case in such discussions, however, he confines it almost entirely to motor vehicles, and the unfortunate pedestrian receives only one half page out of a total of 25 pages. As the writer walks probably 3,000 miles each year, and as the spectator often sees more of the game than the players, some of his comments may be of interest.

Motorists on any particular street or highway can be divided into those who are familiar with the road, and those who are not. The separating islands at crossings, as sketched in the paper, would usually be a help to those familiar with the road or crossing. They would facilitate the channelization of the traffic. For those unfamiliar with the road, however, the reverse would often be the case. As Mr. Kelcey states, a motorist should keep his head up, watch the road 200 or 300 ft ahead, and plan accordingly; but if the traffic is so light that he can see the islands 200 or 300 ft ahead, the islands are scarcely necessary. It is when the traffic is dense and moving fast that they are required; and in that case the motorist, because of the vehicles, cannot see the islands until he is "right on top of them." If he is unfamiliar with the road, he then is likely to become confused and end by stalling the traffic. The complicated channelization approaches to some modern bridges are an example of plans which, while seeming to be perfect on the drawing board, are only too frequently a "headache" in practice.

In 1938 a party of English schoolboys made a motor tour through the United States and Canada. Subsequently interviewed by the press, one of them said that what interested him most was the unhesitating way in which motorists in this country do what they want to do. This was a very gracious way of expressing his astonishment at the antics of the American "road hog." It is the "I want what I want when I want it" spirit, which is a national characteristic, from which no one in America is exempt. When a person is at the wheel, he thinks pedestrians should be abolished and that most of the other motorists are "road hogs"; whereas, when he walks, he thinks that the average motor car is a public nuisance.

There are two practical ways of handling traffic. One is by strict regimentation and the other is by mutual "give and take"—that is, by consideration of the rights of others. In Europe, regimentation is the rule except in England and the Scandinavian countries, where public spirit seems to be effective. With its "rugged individualism" the public in the United States refuses to be regimented. Therefore, the "road hog" must be taught to behave like a gentleman.

With the battleship type of construction, motorcars are becoming like army tanks, and the fatalities to motorists are decreasing. On the other hand, pedestrian fatalities are increasing rather than decreasing. "Shotgun" crossings were threatened in St. Louis, Mo., in 1905, when limited stops for trolley cars were first introduced to speed up the traffic. Those who have seen the unpleasant atmosphere that sometimes develops when accidents occur to pedestrians or children may think that, if such episodes are to be avoided, more public spirit must be developed both in the motorist and in the pedestrian.

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[7] Cons. Engr. (Bury & Waters), New York, N. Y.

The extent to which channelization islands at crossings will improve conditions is uncertain. As Mr. Kelcey states, when speeds increase the accidents increase, and the introduction of islands indiscriminately at crossings may have the effect of increasing speeds. The rush of pedestrians to an expected island refuge, only to find it already crowded, may make conditions worse than if there were no islands; and many pedestrians dislike incurring the risk of being isolated on an island for perhaps several minutes. It is evident, therefore, that such channelization of traffic can in no way be a substitute for "stop-and-go" traffic lights.

The two weak points in the theory of channelization, as so ably explained by Mr. Kelcey, are (1) that it assumes all motorcars are familiar with the arrangements at each crossing, and (2) that it largely ignores the pedestrian.

C. J. TILDEN,[8] M. AM. SOC. C. E. (by letter).—The fundamental principles of keeping highway traffic in line are presented in this paper by Mr. Kelcey. With the rapid development of motor traffic on the highway, increased speeds, necessary changes in roadway design, and other factors, this problem is as important to safe operation of highway vehicles as double tracking was in the development of railways.

The author emphasizes the interesting and important relationship, in so far as motor-vehicle traffic is to be controlled effectively, between psychology and engineering. This relationship has become vitally important in regulating the fundamental factor in highway transport—namely, the driver. In the serious situation with which society is now confronted in the annual loss of thousands of lives and millions of dollars, this cooperation between psychologists and engineers is increasingly desirable.

In the paragraph with the caption "Heads Up," there is brought to mind a query current some years ago in casual groups discussing highway safety. It is, perhaps, of more value in provoking thought than in any definite answer likely to be given. The question was: "How far ahead of the car do you drive when operating on the highway?" The answer, of course, depends on the individual driver, the visibility of the road, the speed, and other factors. The writer tried this question on a number of drivers and several times received the answer: "Why, not at all. I drive where I'm sitting"—or words to that effect; but on further consideration there was always the admission that the driver does look ahead and acts, or is prepared to act, as indicated by what he sees. The actual distance when a concrete estimate was asked for would vary from a few yards in front of the car to "as far as I can see."

The greatest advantage of channelization, perhaps, is the elimination of the head-on collision. Mr. Kelcey's study of relative speeds (Figs. 1, 2, and 3) is interesting and might be extended. Using the condition which he mentions—namely, two cars, one at 45 miles per hr and the other at 40 miles per hr—if their paths are at right angles the relative velocity of *A* to *B* would be about 60 miles per hr in the direction shown in Fig. 51. This is impressive in the case of a collision of cars at a right-angled intersection, a type of accident which is almost as damaging to life and property as the head-on collision. If the speed of each car happens to be 60 miles per hr, the relative velocity, one to the other, is 85 miles per hr, a rate of travel which (as has unfortunately been demonstrated too frequently) is capable of inflicting great damage and loss of life.

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[8] Strathcona Prof. of Eng. Mechanics, Emeritus, Yale Univ.; Pres., Eno Foundation, Saugatuck, Conn.

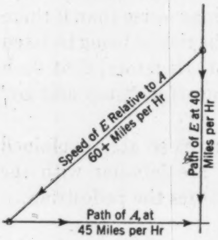

FIG. 51

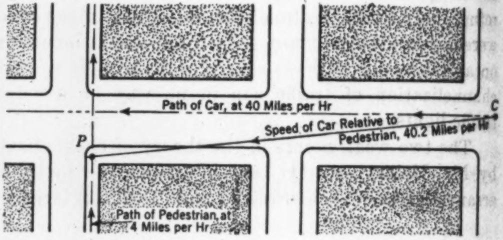
Path of Car, at 40 Miles per Hr
C
Speed of Car Relative to Pedestrian, 40.2 Miles per Hr
P
Path of Pedestrian, at 4 Miles per Hr

FIG. 52

Consideration of the pedestrian problem from a similar angle, even if based on hypothetical (but entirely reasonable) physical conditions, may emphasize some of these difficulties. Assume a broad street or avenue, crossed at regular intervals by others, and a pedestrian, capable of walking 4 miles per hr, whose path leads him across the main avenue (Fig. 52). Assume further an automobile approaching from the pedestrian's right at 40 miles per hr—a speed that is 10 times as fast as the pedestrian's. If the pedestrian, still on the sidewalk at the point, *P*, is 50 ft from the point where his path intersects that of the car, he must give careful (and prayerful) consideration to the car as it passes the point *C*, 500 ft away, or a matter of some two blocks. He must not only sense the car from this distance but he must be able to appraise its speed of approach and govern his own action in crossing the street accordingly. In 1939 the Connecticut Department of Motor Vehicles made a study of highway accidents in which pedestrians had been killed and discovered the startling fact that more than 90% of such victims were not drivers—that is, had never been licensed as operators, and so, presumably, knew little or nothing at first-hand of the behavior and idiosyncracies of automobiles.

In discussing the pedestrian problem, Mr. Kelcey introduces again that combination of psychology and engineering which would seem to be a most promising form of professional cooperation. Eliminating or reducing pedestrian confusion as one definite objective in planning the physical structure of the road is entirely praiseworthy. If this is kept constantly in mind when directing channels are provided, and the channel devices are made to serve also as pedestrian refuges or isles of safety, a long step will be taken toward the goal of pedestrian safety. The pedestrian certainly has rights on the highway and although he should exercise due care, every effort should be made to provide physical aids for his safety.

Mr. Kelcey makes reference to William Phelps Eno and his work in the regulation and control of traffic, which has extended continuously over a period of more than forty years. Indeed, the rotary system of traffic regulation which was first suggested by Mr. Eno about 1905, and described in detail as applied to Columbus Circle in New York City, is a direct ancestor of the principles which Mr. Kelcey has developed. In fact, the germ of the rotary traffic idea was embodied in "Rules for Driving," written by Mr. Eno and adopted as legal traffic regulations by the Police Department of New York on October 30, 1903. Undoubtedly, this set of traffic regulations is the forerunner of all such codes since written because, although it was compiled primarily for horse-drawn traffic before the motor vehicle became a problem in street traffic, subsequent codes, no matter how extended, have used it as a basis.

T. M. MATSON,[9] ESQ. (by letter).—Many elements are developed in this paper which should promote considerable discussion on the part of traffic and highway engineers. Mr. Kelcey is to be congratulated for his work. His efforts should not only influence the design of new roadways, but should also suggest numerous applications at existing "weak spots" in many of the roadways that have been built in the past.

It is apparent that many of the intersections in operation today are failing to carry their traffic loads safely and smoothly because of inadequate channelization mechanisms. It is clear, too, that grade intersections generally are recognized as hazardous locations. Any device that will promote facility of movement with increased safety at the thousands of non-channelized intersections in the roadway system of this nation should be a major factor in the material reduction of the nation's motor-vehicle accident record.

It is evident that traffic islands will need to be introduced into the roadway area if channelization is to be accomplished. Therefore, attention is turned to some of the necessary considerations that are applicable to the placing of such islands in the roadway in such manner as not to increase hazards of operation and yet promote facility of movement.

It must be recognized at the outset that there are many highway engineers to whom any obstruction of any character in the roadway is an anathema. Such feelings are not without some merit and, although the hazard evils of traffic islands are perhaps exaggerated, their virtues as channelizing mechanisms are not at all well understood or appreciated. The problem remains, therefore, as to what can be done toward gaining the advantages of channelization through the use of traffic islands and at the same time avoid the hazards which many people ordinarily associate with them.

Streamlining.—As motor vehicle speeds increase, the need for more and more "streamlining" becomes evident. Although the traffic-approach end of a divisional island may be quite blunt for slow speeds (less than 20 to 25 miles per hr) the need for literally "pulling out" the end, or tapering the approach, of such island becomes acute at high speeds. Just how long a transitional line is required per foot of lateral translation is not well understood. With speeds of 50 and 60 miles per hr, it is quite probable that the lateral translation of vehicles at a rate of greater than 1 to 60 is not too remote from what might be termed an "average natural path of translation." This is quite evident when one notices at what point a motorist begins to veer from his course in clearing an obstacle in his path. At higher speeds, even greater distances of translation are used by the motorist.

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[9] Research Asst., Bureau for Street Traffic Research, Yale Univ., New Haven, Conn.

Research on this point is greatly needed. Until empirical values are determined, engineering judgment must be applied. It is probable that such values as judgment may indicate will fall far short of actual preferential paths of motorists. The need for extreme streamlining of islands under conditions of high speed cannot be stressed too strongly.

Pre-Warning.—As a complement to the streamlining of islands, attention is turned to the need for ample pre-warning of the motorist that he is approaching an island or "void" in the pavement area. This should be accomplished by means of lane markings and other appurtenances. Such center guide lines may be better designed if they begin as ordinary paint lines and increase their severity and tone of "authority" as they approach the end of the island. Thus, beginning with a "no-passing" center line, the next part of such guiding devices may be painted with the usual "cat-eye" forms of pavement buttons. This marker, in turn, would give way to a slightly raised divider strip of material which contrasts in color and texture to that of the pavement. Such a divisional strip would become rougher and more severely ridged as it approached the end of the island proper. In addition to such guiding devices, painted or embossed roadway arrows or other markings may be used in combination forms of guidance which are designed to urge the motorist into a path or channel where the island proper definitely confines the lateral movement.

Location of Approach End of Island.—In general, when a divisional island is introduced into the roadway where speeds are high, every effort should be made to place the actual "approach end" of the island slightly to the left of the apparent center of the roadway. By so doing, the approach end of the island is outside of the normal course of the vehicular stream and additional safety results.

Splitting of One-Way Stream.—When channelization requires that a one-way stream of traffic flow be divided into two one-way streams, it is probable that different methods of island placement will be followed. What has been stated applied primarily to the splitting of two-way traffic into two one-way streams.

When a one-way stream is to be split into two one-way streams, such island of division as may be adopted should be so designed as to leave an area of pavement at the approach end of the island to provide the motorist with a "last chance" turn or decision into either of the two traffic channels. Thus there will be an area of pavement at the crotch of the two ways which will permit the motorist a final opportunity to reach the desired channel. Such pavement should be of different color and texture than that which is used in the normal channels of flow.

Adequate Visibility.—The need for adequate visibility by the motorist of the approach ends of traffic islands cannot be over-emphasized. Here there are two main factors of concern—"sight distance" and "light condition." It is obvious that, under conditions of night, either reflecting or illumination equipment is required to maintain high standards of safety.

The treatment of low-sight distances on the approach to medial divisional islands is a far more difficult problem. Where alinement or crests prohibit adequate safe sight distance, it is necessary either to improve the sight distance or to extend the medial island to a point of tangency or crest where adequate sight distances may be obtained.

Island Curbs.—Because of the importance of adequate visibility, attention is called to the present (1940) developments in curbs for islands. The State Highway Departments of New Jersey and California are experimenting with recessed faces on the island curbs. Such recesses and their materials of construction are especially designed for their light-reflecting properties.

The size and slope of the curbs to be installed for island purposes also merits much attention. There seems to be a trend toward sloping faced curbs of relatively low height. Slopes less than 45° to the horizontal are being used so that the motorist who fails to hold to his channel will not be thrown as severely; nor would his vehicle suffer the damage resulting from high, vertical curbs.

W. W. CROSBY,[10] M. AM. SOC. C. E. (by letter).—Highway engineers have received a most valuable disquisition on the channelization of motor traffic, in this paper by Mr. Kelcey. It is true, of course, as the author states, that the subject is not new; but it is of fundamental importance now, and growing more so yearly. Embryonic channelization may be found in the "white line"—reported as established by white stones set in the middle of the pavement of the 400-yr old road between Cuernavaca and Mexico City, Mex. As Mr. Kelcey states, however, more rigid and actual controls of the flexible motor traffic (by channels) and its temperamental drivers are a growing need, increasing with their numbers and the congestion of traffic.

Something must be done to relieve the congestion, to restore the convenience and efficiency of their paved ways, and to minimize accidents to both vehicular and pedestrian traffic.

For many years the writer has been suggesting some of the points so well explained by Mr. Kelcey, and others have likewise discussed them; but Mr. Kelcey's paper seems to collect and arrange the essentials of his topic more concisely, more forcibly, and more simply and intelligibly than the writer has heretofore observed. Highway authorities can digest Mr. Kelcey's paper most profitably, with all the implications its cogent statements suggest to an active imagination.

The writer again feels that there is still a need for emphasis on two or three points. The author writes of the necessity for wider rights of way and for securing them in advance. The writer has presented this argument in the past[11]; but there seem to be yet many authorities who neglect, fatally, to comprehend the principle, even when entirely new routes are being located, and to realize that the desirable widths can often be had just as cheaply as the insufficient ones taken.

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[10] Cons. Engr., Coronado, Calif.

[11] "Elbow Room," by W. W. Crosby, *Roads and Streets*, October, 1934, p. 362.

Parenthetically, it may be remarked here that not only is it the ultimate development of proper channelization, which is obstructed by insufficient right-of-way takings, but that also the character of the highway and of the development of the adjacent property is degraded or retarded by the establishment of too narrow margins between the high-speed channels ("express-ways" or "freeways") and the private property alongside. Objectionable roadside development, and even the growing aversion to highway noises that drive people from their former homes along a through highway, as well as the depreciation of land values, can be reduced or controlled by proper right-of-way widths and channelization.[12, 13, 14]

Another point the writer would again emphasize is that covered by Mr. Kelcey under the heading, "Human Factors." Motor drivers acquire habits. Their subconscious reactions differentiate them. It is in the public interest to induce, and to cultivate, the habits of good reactions appropriate to the situation. Obscure channelization interferes with (it does not actually destroy) this development. Signs and local ("house" or "ground") rules are only partly effective, or only acceptable in certain cases, especially where the time factor is involved, as is usual in motoring.

Automatic, unavoidable control is infinitely better than signs, rules, or laws toward securing efficient and safe driving.[15] The engineer preaches—for those ends—education, engineering, and enforcement, and there is no question that the last is properly placed. The first two are interrelated, and the priority may be mooted.

To illustrate: The motorist is educated to a subconscious reaction that one turning to the left shall keep to the left in accordance with the usual normal tendencies until that turn is available. Some years ago local regulations in a few places (Washington, D. C., for example) required left-turners to pull off to the right on their green light, stop across the traffic on the cross street, and then swing to the left in the traffic lane with the green light on the cross street. The writer is advised that this practice has been generally abandoned. It seemed abnormal and to require a regrettable amount of mental concentration for those unaccustomed to it.

If "instinct factors" are to prevail, it seems that a left turn demands a left course or inclination preceding that turn. If that is a general rule, then local reversal of it is "out of order"; and yet, on so modern a structure as the "Randalls Island Traffic Circle"[16] traffic turning to the left must first swing to the right. Too many of these special cases will prevent the inculcation of uniform habits. Briefly, the writer would repeat what he has stated elsewhere:[17, 18]

"What looks right may be wrong, but what looks wrong cannot be right."

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[12] Discussion by W. W. Crosby on "The Modern Express Highway," by Charles M. Noble, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 102 (1937), p. 1117.

[13] "The Express Road and the Highway System," by John S. Crandell, M. Am. Soc. C. E., *Civil Engineering*, October, 1937, p. 690; also, *loc. cit.*, December, 1937, p. 859.

[14] "The Parting of the Ways," by W. W. Crosby, *Roads and Streets*, November, 1936, p. 27; also *Civil Engineering*, April, 1938, p. 276.

[15] "Economic and Engineering Problems of Highway Location," by W. W. Crosby, *Transactions*, Am. Soc. C. E., Vol. 91 (1927), pp. 1012 and 1014; also "Safety and the Highway Machine," by W. W. Crosby, *Roads and Streets*, March, 1936, p. 48.

[16] *Civil Engineering*, January, 1940, p. 51.

[17] "Notes on Highway Location," by W. W. Crosby, Gillette Publishing Co., Chicago, 1928, p. 78.

In this connection also may be taken up the question of left turns in intersections as discussed by Mr. Kelcey under that heading and illustrated by Figs. 12 and 13. If the driver adheres to the natural inclination to the left just before the turn (so as to avoid crossing or interfering with following traffic), it may be desirable to "go around the button" for more than one reason. It is in the interest of the extension of present habits to do so. It avoids meeting head-on traffic in the latter's lane on the cross street. It places the left-turner in the proper lane on the cross street (Fig. 40). If the turn "inside the button" is permitted, the tendency is for it to degenerate into the road corner shown in Fig. 14. To allow different practices at different places interrupts the cultivation of the "orderly" procedure desired.

In establishing a rule for the ordinary traffic the highway engineer must exclude from the general consideration the long turning-radius needed by an exceptional vehicle. Often, this vehicle can turn just as well around the button by keeping or swinging to the right before turning left as is done with street-car tracks in some cases of narrow streets.

Channelization would seem to offer a solution that would care for the conflicting interests acceptably in most cases, and some forms are proposed in Figs. 19 and 21. Until it is more widely established, it might be well to insure greater uniformity regarding the left turn, and the writer can only say that he does not yet favor the turns of Fig. 13 except under the direction of a traffic officer.[18]

In Fig. 19 should there not really be two more islands at *B* and at *D*?

Figs. 31, 32, 33, 35, 36, and 37 do not seem to provide adequately for left turns, although they could quite readily be made to do so by channels through or around the islands, which otherwise seem effective. Possibly the author meant his assumption of "no left turns" for Fig. 28 to apply to these also.

For years the writer has been convinced that a widespread, intelligent use of the "rotary system" for relieving objectionable traffic conditions arising from the conflict of traffic streams should be tried. Therefore, he welcomed Mr. Malcher's[6] masterful exposition and development of the principle, which was shown to be of far wider application than to street intersections alone. He now applauds, respectfully, the suggestions and statements of Mr. Kelcey along this line, particularly those under "Divided Highways," "Grade Separations," and "Conclusions."

(One sometimes wonders how much of the present pressure for expensive grade-separation construction comes from producers of the materials and machinery used therefor, and how many times large expense might be avoided by a rotary solution of the problems.)[19]

Channelization might be said to be the "twin"—or, at least, a "baby brother"— of transportation, but its development has not kept pace with the latter. More attention could profitably be paid to channelization so that the two could most effectively perform "in double harness" for the general benefit.

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[18] Report of Committee on Highway Traffic Analysis, *Proceedings*, 9th Annual Meeting of the Highway Research Board, December 12–13, 1929, p. 97.

[19] Discussion by W. W. Crosby on "The Modern Express Highway," by Charles M. Noble, *Transactions*, Am. Soc. C. E., Vol. 102 (1937), p. 1118.

Convincingly, Mr. Kelcey emphasizes the capabilities of channelization. The writer would submit a caution, in view of the tendency to rush to extremes with any remedy—sulfanilimide or war to save the world for the democrats.

Channelization is not a "cure-all." It has its objectionable features, as may be illustrated by the "freeway," which affords protection along its sides to the traffic stream, but at the same time interferes with the use and development of adjacent land.[30] Where width of right of way is unobtainable, the narrowly restricted channels of the freeway may be justified as is an elevated roadway or subway in certain cases. When width can be had (or is reserved by forethought and previous action) the propriety of widening the channels, of increasing their number, or of otherwise providing greater freedom for traffic and development or for other purposes as by a "Pairway," must be recognized.[30, 31]

The broader aspects of channelization have been most interestingly illustrated by John F. Curtin,[22] Jun. Am. Soc. C. E.

Efficient and safe transportation necessitates suitable routing, alinement and grades, and channelization including lane arrangements and provision of closed channels or "pipes" for the vehicles and the pedestrians.

If a road is regarded as a "bridge" between two "bridge-heads" or "terminals," then "channels" of varying degrees of tightness will be useful in collecting and in distributing the bridge traffic.

The writer still believes that the motor (and the driver) should be allowed the greatest freedom of action, on the driver's responsibility, up to the point where another individual may be affected. From then on the needed control should be as automatic as engineering can devise it, as intelligent and habitual as education can cultivate it, and as efficient as enforcement can make it. Channelization will be an important factor in the program.[23]

Julian Montgomery,[24] M. Am. Soc. C. E. (by letter).—The study and analysis of traffic flow are requisites of highway design; and although the operation of a highway system is gaining in momentum, it is not a recently created feature of motor transportation. Mr. Kelcey emphasizes this fact in his references to the research of William Phelps Eno[3] and Fritz Malcher.[4]

As early as 1911, in England,[25] Charles Mulford Robinson declared that "increase in complexity, irregularity, rapidity, and danger of the traffic stream have made transportation needs no longer merely matters of widening streets and untangling old networks to provide for volume." This is an interesting observation in view of the date of publication. It shows the definite thought directed toward traffic flow in the early part of this century. This trend in channelization is even more confining than Mr. Kelcey's reference to caravan routes, etc.

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[30] "Freeways and Pairways," by W. W. Crosby, *Planners Journal*, May-June, 1937, p. 62.

[31] *Civil Engineering*, July, 1937, p. 518.

[22] *Proceedings*, Am. Soc. C. E., November, 1939, p. 1527.

[23] Report of the *Proceedings* of the Hague Cong. (1938) of the Association Internationale Permanente des Congrès de la Route, p. 186 *et seq.*, and p. 275 *et seq.*

[24] State Highway Engr., State Highway Dept., Austin, Tex.

[25] "Width and Arrangement of Streets," Chapter 5, by Charles Mulford Robinson, Engineering News Publishing Co., London, England, 1911.

The word "channel" has been associated primarily with water transportation. By adding "ization," the word has come to be allied with motor transportation. It appears, therefore, that Mr. Kelcey's definition should be expanded to read, "Channelization is an orderly confinement of movement of vehicles to reduce the number of reference points made by overlapping wheel-mark patterns." An addendum to this renovated definition might include reference to friction areas. That reference marks should be kept to a minimum is well known; that the friction area increases as the angle of crossing decreases is also well known. For example, the overlapping space of patterns made by vehicles crossing at right angles is less than the area made by vehicles crossing at 30° and 15° angles.

Mr. Kelcey neglects to inform his readers that the position of the medial strip (as in Fig. 19) governs the turning radius of vehicles with subsequent control of the speed of the vehicle. Medial strips flush with the curb line of the intersecting street provide short radii and slow speeds; medial strips not extending to curb lines increase such radii and turning speeds.

In commenting on the inefficiency of an intersection due to lagging of motor vehicles during interval changes of stop-and-go traffic signal lights, Mr. Kelcey has encouraged the writer in his belief that efficiency of design is measured by highway users in terms of mobility. Design failures—accidents—are not of so much concern to the majority of motor vehicle operators as continual movement. A steady flow, as illustrated in Figs. 47, 48, and 49, of necessity retards the speeds of vehicles, but it does not demand a full stop. In man's desire for mobility, he measures road benefits in continual movement, even if such movements may mean speeds of 10 and 15 miles per hr.

Mr. Kelcey's reference to driving in advance of one's vehicle as "heads up" is what is termed "Road Focus." [26] At speeds of approximately 30 miles per hr a driver's eye is focused only a few feet in advance of his car, but his range of vision extends horizontally several feet beyond the limits of a normal right of way. At high speeds of 60 to 70 miles per hr, the driver's eye is focused far in advance of his car and his range of vision is concentrated on a limited area. Today the demand is for speed. Subsequently, as the engineer designs for speed, he also designs for the visual characteristics of the motor vehicle operator. It appears, therefore, that the engineer carries the responsibility of providing channels for a correct route. This correlates with Mr. Kelcey's suggestion that "drivers will follow the lines of least resistance and use the correct one." If designers are to be guided by such a policy (and the writer thinks that they should), then the modern grade separation commonly called "clover-leaf" is altogether too complicated for the average motor vehicle operator; and although some advocates believe a simpler design should be substituted, it appears that a clover-leaf is the only adaptation providing elimination of both medial and intersectional friction. It seems the driver confusion of this design could be reduced materially by a standard for all highway departments. In providing only the correct route, channelization, when fully adopted, may eliminate both traffic signal lights and warning signs, leaving only directional signs or colored sections of the pavement (Fig. 49).

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[26] "American Highways and Roadsides," by Jac L. Gubbels, Houghton Mifflin Co., publishers, Boston, pp. 22–23.

Mr. Kelcey cites several cases of the failure of laws in adjusting the needs of the motorists. The physical channelization of motor vehicle flow will shift the responsibility of the operation of the highway system solely to highway engineers. This is encouraging because of its practical application as compared to legislative regulations. In Texas, engineers are anxious to develop a speed zone program for obsolete highways; however, they are restricted because of the state speed limit of 45 miles per hr which prohibits posting speeds in excess of this limit. On the other hand, speed studies conducted by the Texas Highway Department[27] reveal that 39% of the operators of passenger cars exceed that speed limit. Speed zoning is handicapped by antiquated laws. Figs. 12 and 13 substantiate the confusion between law and practice. Fig. 12 represents a Texas state law whereas Fig. 13 represents the common practice of most Texas motor vehicle operators. This practice is more advantageous in expediting the safe movement of motor vehicle traffic, as the number of reference points, or crossings, is reduced from eight to four.

The form of rotary traffic,[6, 28] graphically expressed in Fig. 48, is a feature of design which engineers are gradually adding to divided highways in Texas. It is their thought that openings through the medial strip should be governed by the designed speed of the principal, or divided route, in such a manner that traffic may reverse its direction at openings spaced in accordance with accelerating and decelerating distances.

Under "Driving Factors: Merging, Intersecting, and Opposing Movements" Mr. Kelcey declares, "A traffic circle (Fig. 5), with a diameter of several hundred feet, would provide a merging movement of vehicles which would be essentially parallel." First consideration of a rotary traffic circle[29] should be given the merging distance between the egress and ingress legs of the radial roads. This distance is a segment of the circle. It is the writer's belief that by providing sufficient merging distance between radial points, the designer has assured a factor of safety that cannot be ascertained from any other sole feature of design. The length of the segment of the circle depends upon an accelerating distance which would enable the overtaking and passing of vehicles on the arc of the circle between radial points. Such a design approach does not indicate that an increased diameter would provide a merging movement. For example, at the intersection of five roads, the angle of the intersecting roads might warrant the circle to be offset from the center of the intersection in order to provide minimum merging distance for one of the segments of the circle. By such a system, the radius might be kept down to a practical length and still provide the required merging distance. It appears that there is a limit to the radius of a traffic circle. A limit could be established for rotary-circle radii by changing the alinement of one or more of the radial roads in order to provide required merging distance. This could be accomplished according to the principles of design currently used in the decelerating of motor vehicles approaching traffic circles. In investigations of traffic circles in Texas, engineers have found that safe merging distances are more important than any other single design consideration.

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[27] Texas Highway Dept. Information Exchange, Issue 69, May 15, 1939.

[28] "Unfit for Modern Motor Traffic," *Fortune*, August, 1936, p. 85–92, 94, 96, and 99.

[29] "Manual of Instructions, Traffic," Texas Highway Dept. (in preparation).

In 1939, 22% of the motor fatalities in Texas were classified under medial friction whereas approximately 8% resulted from intersectional friction. Definite channelization might have reduced these percentages substantially.

R. M. REINDOLLAR,[30] M. AM. SOC. C. E. (by letter).—The broad scope of Mr. Kelcey's paper, the capable and comprehensive manner in which he makes his presentation on channelization, and the exhaustive information contained therein disclose the time and study which he has devoted to this subject. The control of traffic movement over the highways, involving the direct action of so many individuals in all walks of life, is certain to provoke considerable controversy.

In his calculations of loss of time for traffic passing through intersections controlled by traffic lights, Mr. Kelcey does not take into consideration the compensating factor which, by reason of the fact that cars are "bunched up," allows them to pass through an intersection with closer intervening space between vehicles. At lower speeds, this factor permits a greater number of vehicles to pass a given point within a specified length of time.

The author's description of eddy currents caused by unexpected and irregular movements of reckless drivers is excellent, and should be read by all investigators, as well as trial magistrates, passing on collisions. In many cases one motorist is the innocent victim of actions of another, and not only sustains damage to his vehicle, but is taken to the traffic court and prosecuted for reckless driving, whereas the motorist actually responsible for such a collision is rarely apprehended because of the fact that he is not physically involved.

The author's suggestion to simplify the problem of crossing an intersection by changing the right-of-way law—giving the driver to the left the right of way at intersections, instead of the driver approaching from the right—is a subject on which there will be considerable disagreement. Although the facts in favor of such a change in regulations are ably presented, consideration is not given to the resulting delays to main-line, left-turning traffic that would be caused by ceding the right of way to a motorist approaching from a side road on the left. In addition, it is possible, with four cars from all approaches arriving simultaneously, to have a complete tie-up as described by the author for the right-hand right of way. It is a subject, however, which merits additional consideration and study.

The traffic flow as shown in Figs. 19, 21, 23, and 26 is materially improved by the channelization shown, but more complete control would be obtained by the installation of islands at all approaches, as there is still a possibility of short cutting across opposing traffic lanes on two legs of the intersection. In Figs. 24, 31, and 32, where islands are established for a predominant left-turn movement, they should be wide enough to permit car storage, if right-of-way costs are not prohibitive; and decelerating lanes should be provided so as not to impede through traffic on the main road.

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[30] Asst. Chf. Engr., State Roads Comm., Baltimore, Md.

With reference to the elimination of cross traffic at street intersections on divided highways, which are separated by a narrow center island (see Fig. 48), the advantages of the merging flow might be completely offset by the resulting confusion at the point of left or U-turn between intersections, as decelerating lanes and adequate storage space are not provided. The author describes this treatment in his arrangement as shown in Fig. 49, but the contention is that, unless the features incorporated in this latter development can be applied, crossings at street intersections should not be eliminated.

In conclusion, it should be stated that no highway engineer can read Mr. Kelcey's paper without immediately visualizing many intersections, with which he is familiar, where, by the application of some of the principles as set forth, certain hazards can be eliminated, and a freer movement of traffic can be promoted. It should be read not only from the viewpoint of the economies involved, by reason of savings in time and operating costs through the application of the principles so ably presented, but also for its value as to safety features, which make for reduction in accidents, with their resulting toll in injuries and fatalities.

Irving Mack,[31] Esq. (by letter).—A number of suggestions were not included in the paper which are often helpful to officials who must do the best they can with existing roadways and who, although agreeing with the principles of channelization, find the problem of actual application a little difficult. These matters have to do with the approach to such problems and how the use of stop-and-go signals may be related and applied, where necessary, to a channelization treatment.

The Approach.—Surprizes, ambushes, deceptions, and illusions must be eliminated in the design of any traffic facility so that there will be no unexpected hazard, uncertainty, hesitancy, or confusion under any circumstances of use. Unless it is impossible to avoid, drivers must not be confronted with the choice of more than one of two courses at one time.

In studying the problem of existing pavement area, it is important to lay it out to scale. It is difficult to visualize correctly (as many fail to do), on the ground, the proportions of all aspects of a traffic area or other situation which is more complicated than a simple intersection.

The traffic engineer should study irregular traffic movements in an existing pavement area by observing marks left by the tires of vehicles. If tire marks are not clear, he should throw some sand about the intersection, or dry cement in approach lanes, and let the traffic of the next half hour write its own story.

If the engineer is in doubt as to the correct treatment, tests may be made by one of several methods. In snow country areas the exact shape of islands called for in a design may be filled in with heaps of snow by snowplow crews. A car, motorcycle, barrels, or a heap of sand may be used to test out a proposed treatment. Markers of iron or concrete also may be used for experimental observations, in various positions, for a few hours or days.

A channelizing treatment that has been properly designed and installed invariably exceeds its designer's expectations by working much better than was hoped for. If any confusion develops, at first, it will shortly disappear after local drivers, who have formed fixed habits with respect to the location, become accustomed to it. If it is a good treatment, strangers will know what to do at once.

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[31] Mgr., Electric Signal Div., Signal Service Corp., Elizabeth, N. J.

The matter of public relations should not be overlooked. The engineer should see to it that local drivers know why, when, and how; and afterward he should tell them the results, too.

Stop-and-Go Signals.—The stop-and-go signal is an excellent device when properly used. In fact, its proper use should be greatly extended. Unfortunately, it has often been misapplied. Channelization helps to eliminate the abuses and to assist stop-and-go signals in the performance of their proper functions.

Since channelization permits continuous, safe flow with minimum stoppage, congested conditions, which are often more an evidence of paralysis than of overload, are usually relieved by its application. It is frequently possible, therefore, to correct irregular traffic movements at troublesome intersections to such an extent by channelization that stop-and-go signals, if needed, may be operated as such only during heavy load periods and turned to flashing by time clock, or reverted to manual operation at other times.

When stop-and-go signals are used in connection with a channelization treatment on islands, they signalize the islands whether operating as stop-and-go or flashing. They are also in the most advantageous position to be seen by pedestrians and to control traffic. Traffic control and regulation, pedestrian refuges, and other values are thus all provided at minimum cost by the one treatment.

It is unfortunate, perhaps, that, in his conclusion, Mr. Kelcey stresses so greatly the use of the principles of channelization in new design and neglects to emphasize the advantages of continuous flow and safety to be derived from the application of these same principles to existing roadways.

In the writer's experience, the principles of channelization may be applied to many "Eighteenth and Nineteenth Century" streets and roadways at slight expense and with surprizing results. In fact, such application is often so simple and inexpensive that successful treatments of older roadways frequently await only the effort of studying out the solutions to substitute for or to postpone costly reconstruction.

Bruce D. Greenshields,[32] Assoc. M. Am. Soc. C. E. (by letter).—The very excellence of Mr. Kelcey's paper serves to emphasize the need for accurate design data obtained by careful field studies. It is generally accepted by highway engineers that channelization does increase "the utility and safety of automobile transport by simplifying traffic flow and by eliminating confusions, conflicts, and eddy currents," but there is no agreement as to whether the turning radius at an intersection should be 20, 30, or 40 ft, or some other distance. Vehicle speed, of course, is definitely limited by the turning radii.

The minimum turning radius of cars, trucks, and buses varies from approximately 30 to 50 ft. Joseph Barnett,[33] M. Am. Soc. C. E., gives 28 ft as the minimum turning radius for cars, and 45 ft for buses and trucks. On a parkway limited to passenger cars it may be expected that the turning radii of channels will be less than where truck and bus traffic is permitted; but sharper curves reduce speed and thus limit highway capacity. Furthermore, it must be kept in mind that it is impossible to drive a vehicle in a path composed of simple curves and straight lines. Always the car is eased into a curve by a gradual turning of the steering wheel. The higher the speed, the more gradual is the turning. Too abrupt a turn would invite disaster. Thus, the higher the speed, the longer must be the "easement curve."

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[32] Prof., Highway Eng., Brooklyn Polytechnic Inst., Brooklyn, N. Y.

[33] "Highway Intersections," by Joseph Barnett, *American Highways*, January, 1940, p. 18.

The most advantageous path can be ascertained best by field observations. The camera offers a very good means of making these observations. In interpreting the field data it is important to remember that it is not the mean or average driver's performance that is most important but rather that of the 80 or 85 percentile driver.

The physical factors involved are susceptible of analytical analysis but the human factors are not unless statistical analysis is included. Whether the true spiral, the three-centered compound curve, or, in certain cases, the parabolic curve is most suitable will depend upon the field observations.

The channel of the highway, in contrast to the railroad, must be followed by the willful direction of the driver. This means that the channels must be clearly visible. Light reflecting surfaces and adequate lighting at congested points are necessities for safety.

Unless the dividing islands are to be used for pedestrian safeguards, the writer believes that stop-and-go signals should not be installed on pedestals on these islands. Almost every day somewhere in New York City the base of a signal or light pole is struck and broken by a motor vehicle. In so far as it is possible, the islands should be so designed that cars striking them will not be injured.

T. W. Forbes,[34] Esq. (by letter).—Compliments are due the author, not only for systematizing the wide variety of physical layouts which are met in practical channelization work, but also for introducing a discussion of the very important human factors involved. His conclusion that plans for future designs must take into account driver characteristics and needs are points very well taken. Many engineers have recognized the importance of human factors, but it is of especial significance to see it included in such a paper. In fact, the points mentioned in the paper are worthy of amplification in the following respects:

1. On careful consideration, it appears that the main and outstanding difference between structural highway design and traffic design is that the latter must take account of the human factor.

This difference has been recognized by leading design engineers.[35, 36] Structural design must deal with materials, vehicle weights, and stresses, but need not consider how drivers will use the highway or structure. Engineers working in the traffic field, however, have been forced to recognize the variables introduced by the human factor. Unfortunately, the high variability and difficulty of controlling these factors have led to the general feeling that the human factor is nebulous and impossible to handle.

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[34] Research Asst. (Asst. Prof.), Bureau for Street Traffic Research, Yale Univ., New Haven, Conn.

[35] "The Modern Express Highway," by Charles M. Noble, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 102 (1937), p. 1068; "Thoughts on Highway Design Research as Related to Safety of Vehicle Operation," by Charles M. Noble, *Proceedings*, Highway Research Board, 17, 1937, p. 242.

[36] "Needed Research for the Determination of Sight Distance," by Joseph Barnett, M. Am. Soc. C. E., *Proceedings*, Highway Research Board, 17, 1937, p. 111.

2. If properly approached, however, the human variables can be handled on a definite and practical basis.

It is true that the human reactions that occur in the driving habits of the general public are varied. However, they are subject to certain general laws, and to definite limitations. By combining good physical or engineering research technique with a knowledge of physiology, psychology, and optics, it is possible to determine certain minimum values which will be of practical use in traffic design. The variability of the subject matter must be taken into account by obtaining a sufficiently large number of cases, and a sample representing the driving of the general public. As examples of practical research methods on human factors in traffic design, two of the writer's studies may be cited.[37, 38]

In his paper, the author mentions the general laws of habit, of "least energy," the necessity of pitching instructions for the fairly low average intelligence level, and the fact that driving is directed some distance ahead. These are all good and very pertinent, but can be even more specific.

3. It is not overexaggeration to say that the human factor is the cause of the problem with which this paper is dealing.

The author recognizes this fact in his use of the following expressions throughout the paper: "Reducing uncertainty," "reducing shuttling," "reducing the number of alternate paths," "the necessity of looking two ways," and other similar expressions. To be more specific the writer proposes to examine the fundamental reasons giving rise to this uncertainty and shuttling by the motorist.

In order to analyze the cause of the difficulty, the traffic engineer must understand the kind of mechanism with which he is dealing.

First, the mechanism (the driver) has an angular field of vision of approximately 180°, but his clear vision is limited to a cone of about 5° in the center of this field. He throws this around like a searchlight, and his clear visual attention at any instant is limited to this relatively small field. The driver, then, should be considered as if he were operating his car in the dark by the aid of a searchlight with a bright center beam of 5°, and a very diffuse and low-intensity halo filling the remainder of the 180°.

Second, the "mechanism" requires a certain amount of time to operate. This time period varies in length with the number and complexity of the operations which are involved in the same fashion that the switching time for a call going into an automatic telephone exchange depends on the number of stepping relays to be actuated and the number of exchanges through which the call must go before it arrives at its destination. The simplest operation

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[37] "Overtaking and Passing Requirements as Determined from a Moving Vehicle," by T. M. Matson and T. W. Forbes, *Proceedings*, Highway Research Board, 18, 1938, p. 100.

[38] "Methods of Measuring Judgment-and-Perception Time in Passing on the Highway," by T. W. Forbes, *loc. cit.*, 19, 1939, in press.

is "a simple perception-and-reaction time" which involves an ingoing impulse, a simple central switching operation, and an outgoing impulse. Simple brake-reaction time averages from 0.4 to 0.5 sec. This involves only one reaction, so that no elaborate central station switching is required. However, if the driver must choose between several signals and several courses of action "choice perception-and-reaction time" is involved, in which more complex central switching is required. This is illustrated when the driver sees a red signal ahead, perhaps on a rear car stoplight, and must decide whether to step on his brake or to turn his steering wheel and pass the car ahead. Such choice-reaction times, in fairly simple situations, have been demonstrated to be from 0.75 to 1.0 sec in extent. Under complex situations, they are probably greater than this.

The most complex central station switching, calling into play several different exchanges, occurs when the driving situation calls for a definite skilled judgment on the part of the motorist. This type of reaction is illustrated when the driver must judge the speed of oncoming cars, the speed of his own car, and the speed of the car ahead of him, and decide whether or not it is safe to pass. The time involved in such complex mental "switching operations" or "perception-and-judgment time" is from 2.8 to 3.5 sec.[38]

Now, apply these considerations to the ordinary open right-angle intersection, assuming two 40-ft streets, and a speed of 30 miles per hr. To be conservative, use the one-second "choice-reaction time." This will require approximately 45 ft of travel for a car before the driver will be able even to put his foot on the brake, or start to turn his steering wheel. Therefore, if some action by another car occurs just as he is entering the intersection, he must, of necessity, be across the intersection before he can start to change the course or the velocity of his vehicle—that is, unless he happens to be looking at the vehicle, and the emergency is so obvious as to reduce his reaction to a simple and automatic one, in which case he will be halfway across the intersection.

To make matters worse, examine the 5° cone of searchlight beam, representing his clear visual field. It has been shown that it takes from 0.6 to 0.8 sec to flick this visual searchlight from one point to another and back again.[39] Actual seeing takes place only during the time that the searchlight is resting "fixated" and not moving. The smallness of the cone indicates that in a right-angle intersection each of the three entering ways must be fixated separately when the driver is just entering his own street. Thus, he must give his attention to at least three points at a cost of at least 0.6 sec each; and, if anything unusual happens, his own central switching process may take from about 1.0 sec to as many as 3.0 or 3.5 sec.

When it is considered that each intersection represents not only three entering ways, but two entering lanes of traffic on each, and at least three different general paths which each stream of traffic might take (straight, left turn or right turn), it is apparent immediately that even an ordinary right-angle intersection of two 40-ft streets, with a continuous flow of traffic at about 30 miles per hr, presents a task for the human switching mechanism in which split-second reaction is impossible.

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[38] "A Method for Analysis of the Effectiveness of Highway Signs," by T. W. Forbes, *Journal of Applied Psychology*, 23, 1939, p. 669.

This is the problem which, as the paper so ably points out, is often solvable by channelization. Fundamentally, then, how does channelization solve the problem? First, it localizes conflicts to certain points, small enough to be taken in by the 5° cone of clear vision. Second, it spaces these points so that the human switching mechanism can deal with one of them at a time. Third, it reduces the time necessary for the mental switching process at each point of conflict by reducing the number of choices necessary and by eliminating or simplifying the judgment that must be made. In other words, instead of having to deal with "perception-and-judgment time" the engineer can deal with "choice-perception-and-reaction time," that is, he can reduce the time magnitude from 3.0 sec or more to about 1.0 sec, plus.

Sometimes it has been found in the field that certain channelization has not been effective. If this unsuccessful channelization is submitted to analysis on the foregoing basis, it will be found that channelization succeeds in proportion as it accomplishes the foregoing three objectives. Thus, a definite cause of the problem has been indicated and a definite basis outlined on which the successful solutions may be analyzed in physical terms, but in the light of the knowledge of the "human mechanism."

4. However, the most carefully designed channelization or other traffic design may be made ineffective if certain other human factors are not taken into account by the engineer.

The most important of these is what may be called "mental set or expectation" of the driver. This factor pre-adjusts or sets the "central switching mechanism" to favor certain trunk lines.

The engineer may sometimes forget that, since he is familiar with the layout of a design, he represents such a pre-set system, and consequently he reacts more easily and quickly than does the strange driver who is not thus set. Sometimes the engineer, quite unintentionally, by means of signing, or by means of the preceding alinement of the highway, may even give the strange driver what corresponds to the "wrong number," thus leading him to expect a different type of layout from what he is actually approaching. This results in the "surprises" mentioned by the author. Under these circumstances, it takes extra time for the human mental "switching system" to unscramble the wrong numbers and redial the right ones.

To avoid this confusion, it is important that the engineer submit his new design or redesign to tests from the point of view of the every-day driver. He can do this in two ways. First, he can try to put himself in the place of the strange driver. This is somewhat difficult to do, but after some practice and with the help of friends who are not familiar with the design he may be able to make a fair guess at the stranger's state of mind as he approaches the location. Second, he can measure or observe a large number of drivers on the new design before it is actually installed. This can be done by means of temporary installations, with sandbags or with markers on movable stanchions. It is also possible, for some problems, to select certain test locations which involve similar features, and to make measurements of traffic behavior on these.

From such studies, there will result not only proper design, but also the most effective wording of warning signs and placement of warnings.

5. For any installation on a moderately high-speed route, it is very important that the placement of warnings allows the motorist sufficient time. This time must include not only his own "perception-and-judgment-time" switching operations, but also comfortable deceleration to a speed slow enough for the maneuver.

A time period longer than that discussed herein under Section 3 is thus required, as has also been recognized by the author and by other leading design engineers.[40]

Again, to be specific: In a recent study[38] it was found that perception-and-judgment time, where a judgment of complex speeds was involved, required at least 2.8 sec for the motorist to make up his mind. This value was selected to include about 80% of the motorists. Measurements have not been made at intersections, but a similar judgment-time value probably holds. To this must be added time for deceleration. John Beakey[41] showed in 1938 that as motorists approached a stop sign and decelerated from 50 or 60 miles an hr to 30 miles an hr, they required on the average about 8 to 9 sec. Slowing to 20 miles an hr required from 10 to 11 sec on the average. On this basis, it is apparent that if the installation requires slowing to a turn-off speed of 30 miles an hr, a total of about 12 sec of warning is required, whereas for a turn-off speed of 20 miles an hr to be reached, a warning time of 13 to 14 sec would be needed.

Since the vehicle is decelerating through this distance, a warning time of about 10 sec and 12 sec, respectively, is probably sufficient when converting warning time into distance (for sign design) in terms of the design speed. The deceleration distances of Mr. Beakey's study bear out this conclusion.

From studies of the effect on the motorist of different types of signs,[42] it is possible to predict the necessary height, width, and type of reflectorization of lettered signs to give sufficient warning.

Summary.—1. The author has been commended for his discussion of the human factor in relation to channelization. It has been pointed out that the human factors involved could be stressed even more in the discussion of channelization and traffic design, since they are, in the final analysis, the fundamental cause of the problem which is being treated. However, at the same time it should be emphasized that they are also the fundamental source and reason for the existence of this automobile traffic for which the highways and streets are being designed.

2. The human factors treated by Mr. Kelcey have been recognized by many leading design and traffic engineers, but they have been too often thought of as nebulous and impossible of engineering treatment. It has been shown that this is not the case, and that if the proper approach is made by sound physical research based on psychological and physiological knowledge, these factors can

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[40] "Needed Research for the Determination of Sight Distance at Intersections," by Joseph Barnett, *Proceedings*, Highway Research Board, 18, 1938, p. 76.

[41] "Acceleration and Deceleration Characteristics of Private Passenger Vehicles," by John Beakey, *loc. cit.*, p. 81.

[42] "Legibility Distance of Highway Destination Signs in Relation to Letter Height, Letter Width and Reflectorization," by T. W. Forbes and R. S. Holmes, Jun. Am. Soc. C. E., *loc. cit.*, 19, 1939, in press.

be made definite and specific in a practical way for design purposes. That is, certain minimum and limiting values can be obtained for the guidance of designers.

3. Certain illustrations have been given of definite time values, and corresponding distance values resulting therefrom, which may be considered as the minimum values for traffic design and channelization problems. The studies from which these were derived have been cited.

JAMES S. BIXBY,[43] ESQ. (by letter).—To Mr. Kelcey's able exposition of the advantages derived from vehicle traffic channelization there should be added the warning that every raised island or obstruction provided for vehicle control within the traveled-way is a potential menace to traffic, making it necessary frequently to weigh the advantages of channelization against the hazards that will be introduced by islands in traffic.

Vehicle drivers tend to run straight ahead at the maximum practical speed and any obstructions intermittently introduced in this straight-ahead path are certain to involve danger, in spite of warnings which may seem to be adequate.

At the intersection of a main route and secondary road, islands in the throat of the intersection may be installed and maintained successfully on the secondary road where normal car operation requires the slowing of traffic, whereas the same islands placed on the main road may be the cause of serious trouble, in spite of flashing signals and other safety measures. For the reasons stated, cities and urban areas are more suited to island traffic control because of better lighting and slower traffic, but it is a matter of record that almost every city in the United States has had some experience with traffic-control obstructions that have been installed and later abandoned.

At locations where island control of traffic has proved dangerous, the difficulty has been solved by removing the islands and substituting pavements of a different color from those surrounding the islands. The flat areas of different color, of course, do not provide the same positive control as raised islands but they do develop a very positive influence in controlling the streams of traffic without danger to the vehicles involved.

As suggested by Mr. Kelcey, grade separations are better than traffic circles because the circle, after all, is an obstruction in the path of normal straight-ahead traffic and, therefore, involves danger even if traffic is warned and controlled. For traffic circles in rural areas, adequate entrance drives should be provided so that the speed of main line traffic need not be too sharply curtailed in order to enter the rotary area.

If traffic circles are to be undertaken as a step toward ultimate separation of grades, due consideration must be given to the fact that a circle more than 300 ft in diameter, with good approach drives, can be built on about five acres of land, whereas a separation structure with access drives in each quadrant may require from ten to twenty-five acres, depending on the minimum radius of curvature desired on the access drives. On many traffic-circle installations, the development of business immediately adjacent to the circle will make the cost of later developing a grade separation prohibitive.

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[43] Dist. Engr., Dept. of Public Works, State of New York, Poughkeepsie, N. Y.

GEORGE H. HERROLD,[44] M. AM. SOC. C. E. (by letter).—The ostensible reduction of distance and the actual removal of traffic hazards by improved transportation facilities creating smooth, safe, uninterrupted traffic flow are the ultimate aims of all traffic studies. Mr. Kelcey has taken another step toward these ultimate goals. One great difficulty in providing roadway facilities for the expanding needs of the motor vehicle is that models of cars can be changed every year or two, but streets and highways cannot be changed so often; they must run their life, some twenty years or more, and while each year's construction is the best that advanced engineering knowledge can provide, the conception, design, and product for any year must be accepted and used for many years.

Fortunately, the improvement in motor vehicles that may be anticipated now will be for more safety and comfort, easier operation, better stopping facilities, etc. Building structural faults out of cars and putting new defensive safety elements into cars have been under way for some time. Probably there will be nothing further in the way of speed, but there may be other developments that the engineer's imagination cannot conceive at the present time. However, it would appear that a chance for the engineer to "catch up" is now before him.

One of the essentials in discussing any subject is nomenclature—a system of words describing the various phases of the art or science whose meaning will not change. It has been necessary, through the years, to adopt sentences and words applying to traffic that the layman could understand; but gradually there have been evolving certain expressions that belong essentially to traffic. Mr. Kelcey has chosen his words carefully. He has used the right word in the right place. In fact, he has evolved a language to express, in workmanlike pictures, the needs of this traffic study.

In 1936 traffic experts were introduced to the friction theory of discussing traffic, medial friction, marginal friction, intersectional friction, and internal stream friction. These words ejected a new line of thinking on the traffic problem and probable solutions, and although the engineer may seldom use these words in his daily conversation, they have been added to his vocabulary, and the theories of friction will thus have their influence on current thinking. An engineer may seldom use his calculus on any given problem, yet the method of thinking engendered by a study of calculus is used by the average engineer continuously, unconsciously. So it will be with the friction theories and also with channelization theories.

Traffic planners have played with a number of curious toys during the years —15 miles per hr on city streets, keeping on the right side of the street to turn left, turning around a center instead of the inside turn, speed zones, U turns, barrier curbs, surmountable curbs, light-reflecting curbs, parallel one-way streets, widening two-lane traffic-ways to four-way traffic lanes, free-ways, the law which gives the right of way to the driver on one's right, many center openings on divided roadways, etc.

Railroad trains travel on rails. Switches, turn-outs, cross-overs and signals are provided for a controlled change of movement. Channelization will bring about this same sure type of control.

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[44] Planning Engr., City Planning Board, St. Paul, Minn.

Mr. Kelcey's paper is stimulating, and although it propounds new theories for old conceptions, they are sound. He is to be commended for this interpretation.

Hawley S. Simpson,[45] M. Am. Soc. C. E. (by letter).—It was not so many years ago that an island in the paved roadway was considered as nothing but an "obstruction" to traffic which no intelligent engineer would condone. There was some slight strain of logic behind such reasoning. Most islands constructed about 1925 to 1930 were located, arbitrarily, at the whim of some well-intentioned citizen more concerned with a vista for a decorative flagpole or World War relic than with efficient traffic movement. Islands thus fell into disrepute and practically all were removed.

Then came the day of wider highways which produced some weird intersection movement possibilities and much resultant property loss and personal suffering. The most forward thinking traffic engineers reasoned that "obstructions" could be put to work if properly placed so as to "obstruct" or "block out" undesirable movements and force desirable ones. Although some argued that this theory was only a return to the flagpole day, it was not long before the "channelization bandwagon" was full to overflowing. Today no street or highway can be called truly modern unless designed around barriers that serve to define channels for proper traffic movement.

This trend is most predominantly noticeable on rural highways, but it also finds acceptance on city streets in the use of center strips, pedestrian islands, and channelization islands. The most practical work in this direction has been done in Milwaukee, Wis.[46] Whether or not a direct result of these activities, it remains a fact that, for many years, Milwaukee has enjoyed one of the best safety records of major American cities.

Channelization is successful because it is so simple and natural. It guides each motorist through difficult situations, making it impossible for him to do other than the normal and proper thing. Channelization is positive, exact, and inviolable, easily and readily obeyed by a public which would willingly violate a regulation having the same objective. This is because rules must be over-restrictive to be adapted to the level of the sub-standard driver. Channelization, on the other hand, works equally as well with the sub-standard driver as with the above-par individual. It makes every one a better-than-average driver.

Channelization has many uses. Within the limitations of his paper, Mr. Kelcey could explain only general principles. The writer submits a channelization problem successfully solved in 1925 and functioning as satisfactorily today.

At one of the heaviest traffic intersections in Detroit, Mich. (Second Boulevard and West Grand Boulevard), a substantial volume of left-turning traffic created such congestion that left turns were abolished by order of the police. This created an even worse situation at the adjacent intersections, and so left turns were again permitted—this time around a 6-ft painted traffic circle under the guidance of a police officer. This plan also failed and the writer devised the scheme portrayed in Fig. 53. Straight-through traffic on both Second Boulevard and West Grand Boulevard moves as at any normal intersection. Left-turning vehicles from Second Boulevard, however, take the right-hand lane

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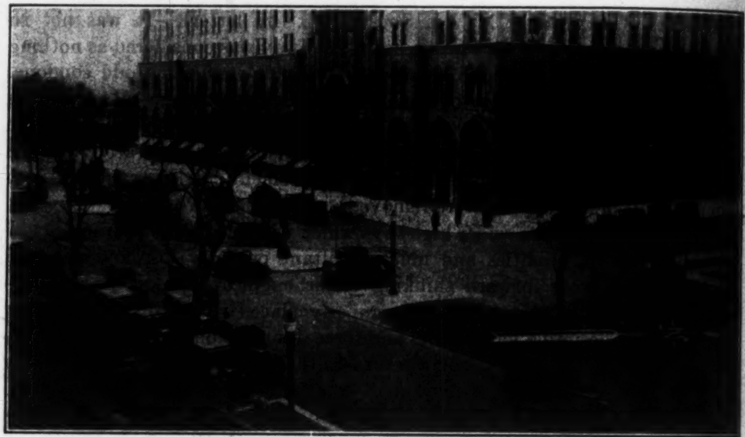
[45] Research Engr., Am. Transit Assn., New York, N. Y.

[46] "Planning Traffic for Easier Enforcement," by Howard F. Ilgner, *Proceedings*, 6th Annual Meeting, Inst. of Traffic Engrs., 1935, pp. 22–31.



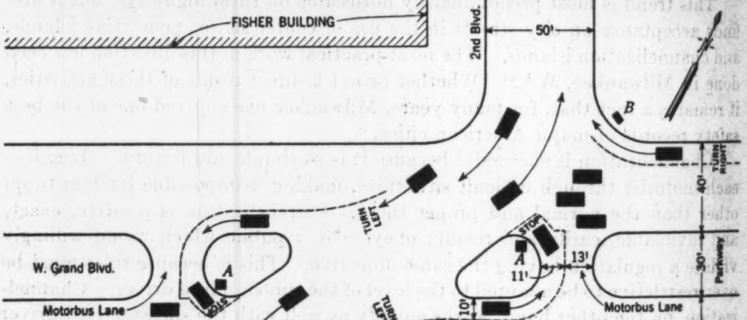

FISHER BUILDING
2nd Blvd.
50'
B
TURN RIGHT
40'
Parkway
30'
40'
TURN LEFT
STOP
A
13'
11'
10'
Motorbus Lane
W. Grand Blvd.
Motorbus Lane
STOP
A
TURN LEFT
0
25
50
100
Scale in Feet
TURN RIGHT
B
50'
2nd Blvd.
GENERAL MOTORS BUILDING
LEGEND
Moving Vehicle
Stopped Vehicle
Paint Lines
A-B
Traffic Signals

FIG. 53.—CHANNELIZATION OF TRAFFIC AT SECOND BOULEVARD AND WEST GRAND BOULEVARD, DETROIT, MICH.

and turn left around the irregularly-shaped traffic island and there await the green signal on West Grand Boulevard. The left turn from West Grand Boulevard to Second Boulevard is similarly made except that it begins in the left-hand lane. In its simplest terms, this arrangement comprises a traffic circle or oval, split in the center so that north-south traffic moves through, rather than around. It is only one concrete example of how a reduction in the paved area, through construction of a traffic barrier, can be used to promote intersection efficiency and safety.

Arthur G. Straetz,[a] Esq. (by letter).—To find methods for the unimpeded movement of vehicular traffic—that is the gist of traffic engineering and its prime purpose. Continuity of traffic is always coincidental to less friction and less accidents. Focusing a critical eye upon Mr. Kelcey's paper, one must find that this is a very important phase of the art, and that it is well developed by the author.

Fundamentals are difficult to establish and difficult to separate from the maze of accumulated material. Channelization is such a fundamental detail, being almost so simple that its importance is usually overlooked; yet it requires a considerable analysis as well as intense study.

There is a possibility that being too close to the subject, one may develop a mental aberration. One sees too closely and one becomes confused. This is slightly noticeable in the section of the paper entitled "Right of Way." The law—natural and adopted—that "the vehicle from the right has the 'right of way'" is not only an established law in the minds of most drivers but is also a word-play easily remembered.

In the end it is uniform application and enforcement that count most unless the subject is against the grain and foolish.

As to the technical layout of existing intersections, two methods may be suggested. One is to cover the intersection with a white powder to produce track patterns and to conform the final layout to the trend of least resistance. The other would be to have movable ramps or curbs which could be adjusted easily.

It is interesting to note the tendency of the author to solve most of his problems without the assistance of auxiliaries such as signs, signals, etc. This is significant because it is an attempt to counteract a quite general assumption that signs or signals are the final remedy for the solution of a traffic puzzle. Although the author has interests in mechanical aids to traffic he tries to be logical and succeeds in being so. This is quite in line with the solid professional standard exemplified by the paper, which deserves the attention of any national organization dealing with the traffic problem.

It is quite doubtful whether any other profession as young as that of Traffic Engineering has as complicated and as difficult a task as that of the traffic regulation and the prevention of traffic accidents. He who has been able to keep himself in line despite all the difficulties arising from being on the semi-commercial end of the "game" and yet maintaining a fine professional attitude, as in this paper, is certainly to be commended.

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[a] With Kopp Glass, Inc., Chicago, Ill.

Burton W. Marsh,[48] M. Am. Soc. C. E. (by letter).—Not so many years ago, there was extensive opposition among highway engineers to proposals of traffic engineers for the use of islands in roadways. Islands were considered obstructions and no obstruction was viewed as desirable. Fortunately there has been a marked degree of change in such viewpoints, and many, if not most, leaders in highway engineering recognize that various conditions warrant roadway islands. It is becoming generally agreed that warranted islands, properly designed and maintained, benefit motorists and pedestrians.

Mr. Kelcey's splendid paper will do much to clarify thinking on the subject and to encourage greater use and less misuse of such aids to more efficient, convenient, and safe traffic movement.

Greater emphasis seems warranted in forewarning drivers of the presence and location (especially as to physical limits) of islands. This need is especially great at night and under other unfavorable conditions, such as snow, fog, and rain. The better designed and located the island and its "approach end," the less is such need; but the need always exists, for a driver may attempt to overtake and pass a large truck or may undertake some wrong maneuver which he would not make if he had adequate forewarning concerning the island ahead.

Effective lighting of islands is essential, with special attention to approach ends. Such lighting should not only warn the motorist that something unusual is ahead, but should also floodlight the island end so that the driver can see clearly what is ahead in plenty of time to make any necessary adjustment in his path, and to avoid starting any ill-advised maneuver. The higher the speed, the sooner the driver should be guided, because at high speed changes in lateral position require considerable distance in the direction of travel. Other aids to drivers approaching islands include guide lines on the pavement, gradually rising physical guides of distinctive color and with gradual sloping sides, and island curbs so designed as to indicate their position by light reflected back to the driver's eyes.

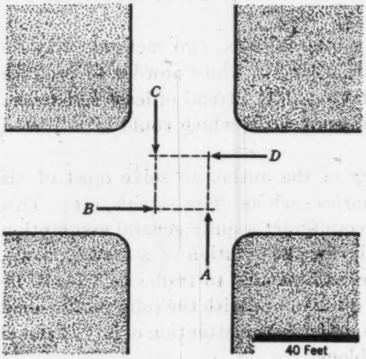

Fig. 54.—Tie-Up Positions of Four Vehicles if There Were a Left-Hand Right-of-Way Rule

Mr. Kelcey is to be commended for not omitting today's most neglected factor in traffic—the pedestrian. All island design should take fully into account the problems, rights, needs, and convenience of the man afoot.

One illustration of the need for more attention to the pedestrian is involved in Mr. Kelcey's discussion under the heading "Driving Factors: Right of Way." Considering vehicles only, Mr. Kelcey states (referring to Fig. 16) that *B*, on his left, is driver *A*'s "first and immediate problem on entering the intersection." A study of 1,008 pedestrian accidents occurring at 76 intersections in Philadelphia (not including turning vehicles) showed that 76% of the pedestrians were struck before they reached the middle of the roadway. Nearly one half (45.9%) were so struck on the near side of the intersection. If, as is customary, the word "intersection" is so defined as to include the cross walks, driver *A*'s first consideration should be for pedestrians.

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[48] Director, Safety and Traffic Eng. Dept., Am. Automobile Assoc., Washington, D. C.

The aforementioned Philadelphia study also gives support to the present right-hand, right-of-way rule, because the 45.9% situation will be improved if drivers approach intersections slowly enough and attentively enough—in this case, attentive to the right especially—so that they can avoid even much misjudgment and lack of attention on the part of those crossing afoot. Of course, one cannot "look two ways at once."

There is another point favoring the right-hand right-of-way which relates to the vehicular situation entirely. Since it is natural and perhaps instinctive for the thinking driver to give attention, as between cross-vehicular-traffic directions, first to the left because that is the cross-pathway which he must meet first, is there not merit in requiring him (by the right-hand, right-of-way rule) to give proper attention also to the right before he (so quickly) enters the zone of that possible vehicular conflict?

In his Fig. 16, Mr. Kelcey shows how it is possible, with the right-hand, right-of-way rule, for a "tight" tie-up of four vehicles to occur. A companion diagram could be drawn for a similarly possible tie-up of four vehicles if there were a left-hand, right-of-way rule (see Fig. 54). Admittedly, the physical positions of cars would make it easier to clear up the tie-up in the latter case.

VIRDEN A. RITTGERS,[49] ASSOC. M. AM. SOC. C. E. (by letter).—Channelization, before it can be applied, must be "sold" to the proper officials in many instances. There is one case of a municipality spending thousands of dollars in removing a double-track street-car line and center parkway from a wide major artery and then being urged to paint a 6-in. white center line to keep vehicles on the proper sides of the street. A channelization opportunity was overlooked entirely.

Mr. Kelcey's fine paper will contribute, with certainty, to this problem of selling the principles of channelization. In fact the writer has already found the paper of practical assistance in this respect. Profusely illustrated and written in a style which is impelling through use of clear, concise statements and a splendid choice of words, it is a pleasure to read.

The author deals exclusively with turning movements so far as intersection channelization is concerned. The writer has observed another faulty driving habit which is so pronounced in some urban communities as to be of real import. Fig. 55 illustrates this habit in the approach to a signalized intersection. Driver *A*, in a hurry and not content to take his place behind the vehicles waiting at a red light, pulls to the left side of the roadway and "hogs" his way to the front of the line. Here he constitutes a hazardous interference to cars turning from the cross street and to opposite-direction traffic when the signal changes. Here again channelization with a center island discourages and practically prevents the wrong movement. Seldom will even the unsportsmanlike driver pull to the left half of the roadway when there is a structural definition between the left and right sides. Usually the trespassers of this type are "home-town" drivers in a rush to get somewhere during heavy traffic periods. A few drivers will pull over to the left, not knowing the island is ahead, until they have lost their places in line. In such instances they will usually hold up behind the island until they can "horn in" where they belong.

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[49] City Traffic Engr., Police Headquarters, San Antonio, Tex.

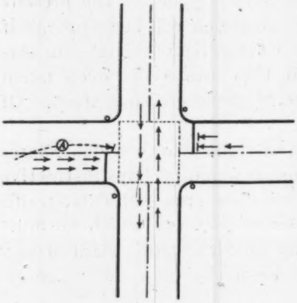

Fig. 55

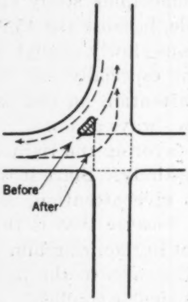

Fig. 56

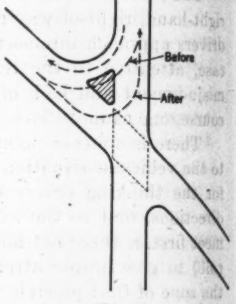

Fig. 57

Mr. Kelcey's paper, dealing primarily with the principles of channelization, contains examples of only those intersection approaches which are sufficiently wide to accommodate center islands. Many traffic engineers, when it comes to doing the job, will find so many narrow street complications that it seems worthy of mention. Figs. 56 and 57 illustrate typical urban examples of roadways that are too narrow for center islands but where "corner" channelization may still be applied to regulate faulty left turns (to a substantial extent if not fully) and to facilitate pedestrian crossings and sign and signal placement. Many problems of this type have come about during recent years through the "corner cutting" projects of many cities, particularly under the government relief programs. The writer has in mind one instance in which 40-ft and 50-ft radii were standards on wide residential streets having pavements only 26 to 30 ft wide.

Rows of mushroom-type, concrete traffic buttons have been used in a number of instances to meet the narrow street (and perhaps the cost) problem in channelization. This plan, being a compromise (which is never right) between the feeble paint line and the raised island, is fairly satisfactory for many jobs. Its greatest weakness lies in the difficulty of making the installation visible, thus increasing the "trapping" effect on drivers.

The principles of channelization, as described by Mr. Kelcey, are certainly sound principles. Help for the pedestrian is of major importance (although treated only briefly by the author for reasons which he gives). A subsidiary advantage lies in the increased facility for placing necessary signals and signs at exactly the right spots. This advantage will serve to offset the element of confusion (often more imaginary than real) which might be claimed for some complicated layouts. As a general rule, when traffic is light and speeds higher, the roadway situation ahead is more easily seen and understood; and when traffic is dense, speeds are lower and greater visibility distances are not so essential.

Guy Kelcey,[50] M. Am. Soc. C. E. (by letter).—To all who made substantial contributions in presenting this most interesting subject, both in its original preparation and in the constructive discussion that followed, the writer wishes to express his appreciation. Some of the questions raised have been explained in a supplementary paper.[51]

Mr. Waters suggests that two weak points in the theory of channelization are: (1) It assumes that all motorists are familiar with the arrangements at each crossing; and (2) it largely ignores the pedestrian. The writer feels that proper channelization should simplify crossing problems to the point where "what to do next" is obvious. If a motorist is in doubt, at any time, the channelization treatment is not a good one or should not have been attempted until other conditions, which prevent a proper treatment, have been corrected.

Channelization is not in itself a cure-all. It often solves the problem of the pedestrian by eliminating trouble-causing factors. Its primary purpose is to solve the problem of the vehicle; thereafter, greatest simplicity and certainty of vehicle movement having been attained, further steps to solve the pedestrian problem, if needed, become simpler and easier. If, then, channelization does not solve the difficulty in certain instances, it paves the way for the best and simplest solutions of pedestrian and other remaining problems.

Mr. Crosby asks if there should not have been two more islands shown in Fig. 19. Islands in two opposite throats of a right-angled intersection usually prevent troublesome irregular vehicular movements within an intersection. If pedestrians are numerous the two islands are often best installed in the two opposite intersection throats which carry the greatest pedestrian load. If pedestrian flow is heavy in all directions, islands may be placed in all four throats for their protection and for more complete vehicular control. If no pedestrians are involved, islands are usually installed in opposite throats of the roadway carrying the least traffic unless there is need, as is often the case, to divide the main flow.

Mr. Crosby also questions if, in Figs. 31, 32, 33, 35, 36, and 37, provision should have been made for left turns. Roadways, which intersect at acute angles, usually have cross roadways behind the intersection, in each direction, which can be used and which eliminate the need for the left-turn provision. However, the solutions shown in Figs. 31, 33, 35 (if enough roadway space permits), and 37 may be adjusted to permit left turns. On the other hand, it is often desirable, on the grounds of expediting and safeguarding the greatest number, to prohibit or block out the limited demand for left turns at intersections of this kind.

Particular reference is made to the discussion of Professor Forbes who has made a new and most valuable contribution in his very clear suggestions for measuring psychological and physiological values.

Discussions by Messrs. Tilden, Matson, Reindollar, Montgomery, Mack, Greenshields, Bixby, Straetz, Simpson, Herrold, Marsh, and Rittgers have all added constructively and substantially to the subject.

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[50] Signal Service Corporation, Elizabeth, N. J.

[51] "Traffic Channelization Methods," by Guy Kelcey, *Civil Engineering*, October, 1940, p. 645.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 2108

NORRIS DAM CONSTRUCTION CABLEWAYS

By R. T. Colburn,[1] M. Am. Soc. C. E., and L. A. Schmidt, Jr.,[2]
Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. J. S. Foster, Gordon H. Bannerman, Walter F. Weber and Blair Birdsall, G. E. Cate, Adolph J. Ackerman, and R. T. Colburn and L. A. Schmidt, Jr.

Synopsis

The development and application of thrust-wheel type traveling cableways and the detailed features of design at Norris Dam are here explained. The design features were checked under field conditions by tests which are described, analyzed, and compared to the design figures. In general the tests check the original computations closely, but in some instances they pointed to opportunities for further study and precautions to be used in the application of theoretical formulas.

Preliminary Investigation

Norris Dam was the third construction project to use heavy duty cableways supported on traveling towers specially constructed to take the horizontal thrust entirely through a set of horizontal thrust wheels. The first was the Madden Dam in Panama, with a span of 1,325 ft. Soon after that successful use, three similar installations were made at Hoover Dam, with spans ranging from 1,365 ft to 2,575 ft.

Previous to these installations, when traveling type cableways were used the towers were usually mounted on the heaviest kind of railroad trucks, and the entire horizontal thrust was carried on the wheel flanges. As greater projects were undertaken, loads were increased and larger sized concrete buckets were used, requiring an increased number of vertical wheels to support the towers and to carry the horizontal thrust. The excessive flange loads caused a great many broken axles, and shutdowns while repairs were being made, with additional expense. The numerous axle failures on the Owyhe Dam cableway were typical of this experience.

For the Norris Dam operations 6-cu yd concrete buckets were contemplated. With a cableway span of 1,925.5 ft, this load would cause a horizontal thrust on

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Note.—Published in December, 1939, *Proceedings*.

[1] Constr. Plant Engr., TVA, Knoxville, Tenn.

[2] Care, TVA, Hales Bar Dam, Guild, Tenn.

the towers of more than 200 tons, which is too great to carry on wheel flanges. A thrust-wheel type tower was clearly indicated under these conditions. To handle concrete in smaller buckets would have required a slower construction schedule, and the cost of placing the concrete would have been correspondingly higher. The two Norris Dam cableways together placed 1,005,000 cu yd of concrete and handled more than 68,000 tons of other materials during the construction of the dam.

The topography on each side of the river was such that runways could be built economically. The right bank of the river facing downstream was a rocky bluff, sloping about 45° with a suitable location for the head-tower runway about 105 ft above the roadway on top of the dam. The center part of this runway was a rocky shelf which was excavated from the bluff and which sloped off at the extremities, necessitating the use of a heavy steel trestle at each end of the runway. The tail-tower runway on the left bank was constructed entirely on earth fill. The topography permitted this fill to be built to the same elevation as the roadway on top of the dam, and later to be used as part of the permanent access road.

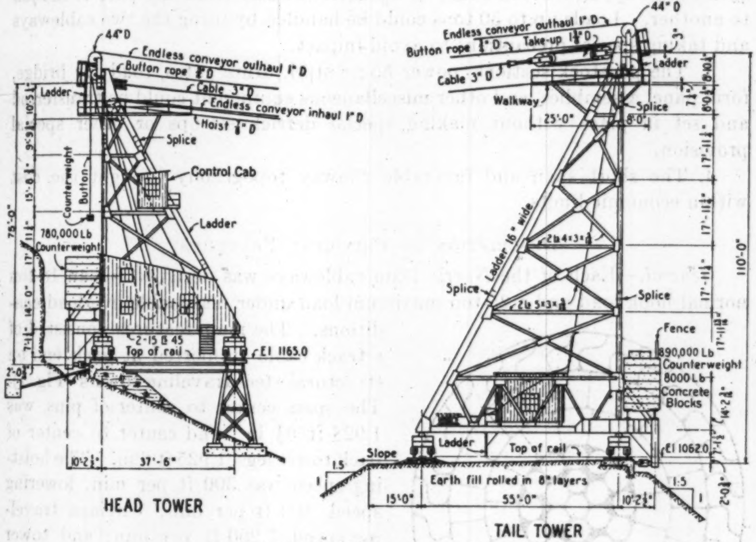

44" D
Endless conveyor outhaul 1" D
Button rope $\frac{7}{8}$" D
Cable 3" D
Endless conveyor inhaul 1" D
Hoist $\frac{7}{8}$" D
Ladder
Splice
Control Cab
Counterweight
Button
Ladder
780,000 Lb Counterweight
2-15 [45
Top of rail
El 1165.0
75'-0"
10'-5$\frac{1}{2}$"
9'-9$\frac{1}{2}$"
15'-8$\frac{1}{2}$"
11'$\frac{1}{2}$"
17'
14'-0"
7'-1$\frac{1}{2}$"
2'-0$\frac{1}{2}$"
10'-2$\frac{1}{2}$"
37'-6"
HEAD TOWER
44" D
Endless conveyor outhaul 1" D
Button rope $\frac{7}{8}$" D
Take-up 1$\frac{1}{8}$" D
Cable 3" D
Ladder
Walkway
Splice
25'-0"
8'-0"
Ladder 16" wide
2 Ls 4×3×$\frac{3}{8}$
2 Ls 5×3×$\frac{3}{8}$
Splice
Splice
Fence
890,000 Lb Counterweight
8000 Lb Concrete Blocks
Ladder
Top of rail
El 1062.0
Slope
1.5
1
Earth fill rolled in 8" layers
1.5
1
15'-0"
55'-0"
10'-2$\frac{1}{2}$"
3"
8'-10$\frac{1}{2}$"
6'-0$\frac{1}{2}$"
7'$\frac{1}{2}$"
17'-11$\frac{1}{2}$"
17'-11$\frac{1}{2}$"
17'-11$\frac{1}{2}$"
17'-11$\frac{1}{2}$"
14'-2$\frac{1}{2}$"
7'-1$\frac{1}{2}$"
110'-0"
2'-0$\frac{1}{2}$"
TAIL TOWER

FIG. 1.—DESIGN DETAILS OF THE TOWERS

The thrust-wheel type cableway (Fig. 1) permits the entire horizontal thrust to be placed on the back side of the runway; whereas, on the vertical flanged-wheel type, as much as 80% of the thrust is taken by the front edge under full load. The use of thrust wheels at Norris Dam permitted the earth-fill runway at the tail tower to be built 10 ft narrower than the conventional type would have required. At the head tower (Fig. 1), it would have been dangerous to permit thrust on the front of the runway, first because this was directly on the edge of the bluff and especially because of the trestles at the ends. For these reasons, towers with horizontal thrust wheels at the back of the runways were chosen.

This type of design later proved its effectiveness when a cave-in occurred under the head-tower runway. It was conceivable that at this time one of the cableways might have been damaged considerably had not the horizontal thrust been transmitted to the back of the runway where it was solidly anchored into the hill. The runways were constructed so that all concrete in the main dam, the power house, and the apron could be placed from the cableway. The only place that was inaccessible was the small section of core wall which passed under the tail-tower runway on the left bank. This yardage was small, and special provisions were made for it.

Before the final decision was made other methods of placing concrete were also considered but these were found to be neither as economical nor as practicable for this particular project. The following advantages seemed to justify a cableway installation:

1. Clear access was provided to all parts of the work at all times without the inconvenience of trestles and derricks.
2. A hoisting capacity of 18 tons was available everywhere on the job from each cableway, and a load could be transferred easily from any part of the job to another. Loads up to 50 tons could be handled by using the two cableways and taking special precautions to avoid impact.
3. The penstock sections, power house steel, drum gates, roadway bridge, form panel assemblies, and other miscellaneous equipment could be transferred and set in place without making special derrick set-ups or other special provision.
4. The short span and favorable runway topography brought the cost within economic limits.

Description of Principal Features

General.—Each of the Norris Dam cableways was designed for an 18-ton normal hook load and a 25-ton maximum load under reasonable overload conditions. The main features consisted of a track cable supported at each end on structural steel traveling towers (Fig. 1). The span, center to center of pins, was 1,928 ft 0½ in.; and center to center of back tower legs, 1,925 ft 6 in. The hoisting speed was 300 ft per min; lowering speed, 400 ft per min; carriage traveling speed, 1,200 ft per min; and tower traversing speed, 50 ft per min. Table 1 gives a summary of the principal physical data.

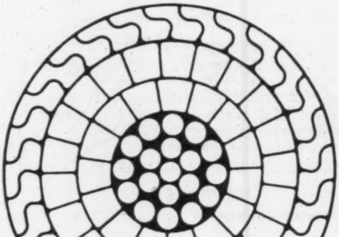


FIG. 2.—CROSS SECTION THROUGH 3-IN. TRACK CABLE

Main Track Cable.—The 3-in. diameter locked coil cable (Fig. 2) was socketed at both ends with roller-bearing swivel sockets. At the head-tower end the bearings were seated against a cross-bar which closed the open end of a heavy U-bolt, the closed end of which bore against the main pin in the tower.

TABLE 1.—Physical Data; Norris Dam Cableways

Item No.	(a) Towers	Head tower	Tail tower
1	Weight of framing, in tons	126	130
2	Total weight (exclusive of counterweight), in tons	227	180
3	Weight of counterweight, in tons	390	445
4	Vertical height from track-cable connection to track, in feet	75	110
	Distance, Center to Center, of Trucks, in Feet:		
5	Parallel to track	32.05	38.05
6	Perpendicular to track	37.50	55.00
	Counterweight Blocks (Each Block):		
7	Dimensions, in feet	10 × 3 × 2	10 × 3 × 2
8	Average weight, in tons	4.5	4.5

	(b) Trucks	Main	Thrust
	Number:		
9	Trucks per tower	4	2
10	Double-flange wheels per truck	8	4
11	Diameter of wheels (double flange), in inches	27	24
12	Weight of rail, in pounds per yard	131	131
13	Distance, center to center of truck wheels, in feet	4.96	3.00
14	Diameter of wheel axles, in inches	7	5.44
15	Weight per truck, in tons	9.1	2.6

	(c) Carriage, Fall Blocks, and Fall Rope	Dimensions
16	Total weight, in tons	11
	Carriage Sheaves:	
17	Number	12
18	Tread diameter, in inches	24
19	Number of hoist-line sheaves	4
20	Tread diameter of load-line sheaves, in inches	35.01
	Fall Block:	
21	Number of sheaves	2
22	Tread diameter, in inches	44
23	Capacity of hook, in tons	25
24	Number of parts on fall line	4

	(d) Head-Tower Machinery	Diameter	Length	Weight
25	Main hoist: Three-drum; all-electric; hoist drums, 49-in. diameter by 44 in.; endless drum, 45-in. diameter by 44 in., with removable spool, 52 in. by 20 in.; nominal capacity, 18 tons	....	....	....
	Weights of Hoist, in Tons:			
26	Endless drum	....	....	12.5
27	Hoist drum	....	....	10
28	Spare hoist drum	....	....	10
29	Stands, etc.	....	....	4.5
30	Bed plate	....	....	5
31	Main hoist motor: 400-hp, 2,200-volt, three-phase, 60-cycle, 600-rpm, wound-rotor, induction motor; total weight (with bed plate), in tons	....	....	5
32	Air compressors: Two compressors per head tower; capacity, 50 cu ft per min; each driven by an 8-hp, 440-volt, three-phase, 60-cycle, motor; total weight (two compressors and receivers), in tons	....	....	2.5
33	Air receivers: Two welded tanks, bell ends; capacity, 24 cu ft; 24-in. diameter by 90 in. long	....	....	....
34	Tower-moving winches: One for each tower; single-drum electric; drum, 21-in. diameter by 26 in., with removable spool 32.5 in. by 12.75 in.; spool driven by 75-hp motor; 440-volt, three-phase, 60-cycle, 900 rpm, wound-rotor induction; weight of winch and motor, in tons	....	....	5
	Miscellaneous Weights, in Tons:			
35	Electrical equipment, head tower	....	....	3
36	Electrical equipment, tail tower	....	....	3
37	Sheaves on top of head tower	....	....	1.5
38	Sheaves on top of tail tower	....	....	1
39	Track cable: locked coil (weight in pounds per foot)*	3	1,900	22.2
40	Endless cable (diameter in inches and length in feet)	1	4,110	....
41	Hoist cable (diameter in inches and length in feet)	⅞	2,500	....
42	Button line (diameter in inches and length in feet)	¾	2,025	....
43	Take-up cable (diameter in inches and length in feet)	1⅜	600	....
	Traversing Cable (Diameters in Inches and Length in Feet):			
44	Tail tower	⅞	2,100	....
45	Head tower	⅞	2,200	....

* 3-in. locked coil; distance between sockets, 1,900 ft.

At the tail-tower end, a similar arrangement was used except that two eye-bolts were used instead of the U-bar. The eye ends of the bolts were connected to the track cable take-up through a pin. The roller-bearing mountings permitted the track cable to be turned so that the wear would be distributed evenly over its entire circumference. When the loaded bucket was being transported the carriage was not run closer to either support than 140 ft, but when the hook was empty the carriage was permitted to approach close to either tower.

The take-up cable for the track cable consisted of a $1\frac{3}{8}$-in. regular lay cable, dead-ended to the top of the tail tower (Figs. 1 and 3). This cable passed through a series of sheaves forming a 10-part line, then back over a sheave in the tail tower and down to the base of the tower where it was made fast. The take-up was fastened to the tower by a main pin and attached to the track cable at the other end, as previously described. The sag of the main track cable, 0.055 of the span, was maintained within 5% by means of the take-up.

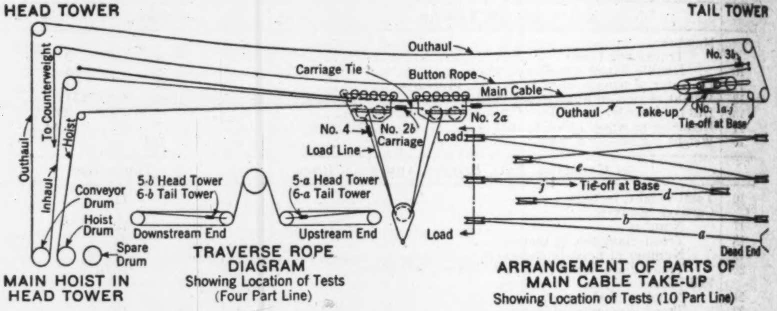

HEAD TOWER
TAIL TOWER
Outhaul
Carriage Tie
Button Rope
Main Cable
No. 3b
Outhaul
Take-up
No. 1a-j
Tie-off at Base
No. 2a
No. 4
No. 2b
Carriage
Load
Load Line
To Counterweight
Hoist
Outhaul
Inhaul
Conveyor Drum
Hoist Drum
Spare Drum
5-b Head Tower
6-b Tail Tower
5-a Head Tower
6-a Tail Tower
Downstream End
Upstream End
Load
e
j
Tie-off at Base
d
b
a
Dead End
MAIN HOIST IN HEAD TOWER
TRAVERSE ROPE DIAGRAM
Showing Location of Tests
(Four Part Line)
ARRANGEMENT OF PARTS OF MAIN CABLE TAKE-UP
Showing Location of Tests (10 Part Line)

FIG. 3.—ROPE DIAGRAM. NUMBERED AND LETTERED POINTS SHOW LOCATION OF TESTS

Operating Cables.—The endless cable, also called the conveying cable, was of 1-in. diameter plow steel. In the original installation regular lay was used which was later changed to flattened strand due to excessive wear on the conveyer drum and the cable. This cable (Fig. 3) is what its name implies—a continuous line fastened at both ends to the carriage. One part of the cable, called the "outhaul" cable, was attached to the front of the carriage; it passed over two sheaves in the top of the tail tower, then spanned the entire distance to a sheave in the top of the head tower, then down to the conveying drum on the hoist where it made four and a half turns around the drum. Continuing back up to a sheave in the top of the head tower and out to a dead end on the rear of the carriage, this other part is called the "inhaul" cable. Rotations of the conveying drum moved the carriage back and forth on the track cable.

The hoisting cable was of $\frac{7}{8}$-in. diameter improved plow steel. This cable (Fig. 3) was dead-ended on the carriage, passing through the sheaves on the carriage and hook block to form a 4-part line, from which point it was returned over a sheave at the top of the head tower and thence to the hoisting drum. By turning the hoisting drum the load was raised or lowered. The hoisting and endless drums were geared together so that when the carriage moved back and forth on the track cable the load remained at the same elevation—it was impossible to hoist and convey at the same time.

The button line consisted of a $\frac{3}{4}$-in. cable dead-ended on the tail tower, spanning across to a sheave in the head tower. After passing over this sheave, the cable was counterweighted with a concrete block weighing approximately

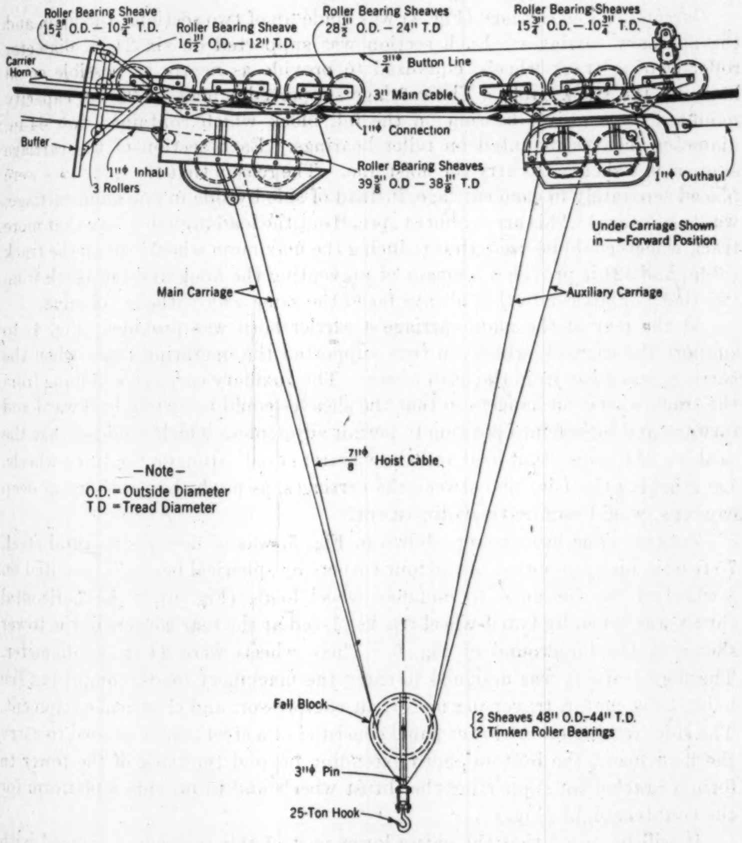

Roller Bearing Sheave
$15\frac{3}{4}''$ O.D. – $10\frac{3}{4}''$ T.D.
Roller Bearing Sheave
$16\frac{1}{2}''$ O.D. – 12'' T.D.
Roller Bearing Sheaves
$28\frac{1}{2}''$ O.D. – 24'' T.D
Roller Bearing Sheaves
$15\frac{3}{4}''$ O.D. – $10\frac{3}{4}''$ T.D.
Carrier Horn
$\frac{3}{4}''\phi$ Button Line
3'' Main Cable
1''ϕ Connection
Buffer
1''ϕ Inhaul
3 Rollers
Roller Bearing Sheaves
$39\frac{5}{8}''$ O.D – $38\frac{1}{8}''$ T.D
1''ϕ Outhaul
Under Carriage Shown
in → Forward Position
Main Carriage
Auxiliary Carriage
$\frac{7}{8}''\phi$ Hoist Cable
–Note–
O.D. = Outside Diameter
T D = Tread Diameter
Fall Block
2 Sheaves 48'' O.D.–44'' T.D.
2 Timken Roller Bearings
3''ϕ Pin
25-Ton Hook

FIG. 4.—CARRIAGE ASSEMBLY

6,000 lb. At regular intervals throughout the span buttons were fastened to this cable. These were so located and were of such a size that each one intercepted a carrier as the carriage passed along the span. These carriers supported the hoist cable and inhaul cable, eliminating deep sags or slack in these lines.

Originally the design of the cableway called for another operating rope for dumping the bucket from the cableway tower. An extra drum in the hoist and extra sheaves in the carriage were provided for this purpose; but this method of dumping was considered too dangerous for continuous service because it would have been possible to dump a bucket of concrete accidentally in mid-air. Therefore, use of this cable was abandoned, and the extra drum was used instead as a spare hoisting drum. These various ropes adjacent to the towers are shown in Fig. 1 and their relations to the operation of the cableway are indicated diagrammatically in Fig. 3.

Carriage.—The carriage (Fig. 4) was made up of two sections, the main and the auxiliary carriages. Each section was supported on six 24-in. diameter roller-bearing track wheels, equalized to provide as nearly as possible equal loads on the track cable. The carriage supported a hook of 25-ton capacity mounted on a swivel bearing on the fall block which contained two 44-in. diameter sheaves mounted on roller bearings. Each section of the carriage supported a sheave to carry the hoist line. The reason that these sheaves were placed separately in each carriage, instead of side by side in one main carriage, was two-fold: (1) This arrangement spread out the load in such a way that more track wheels could be used, thus reducing the maximum wheel load on the track cable; and (2) it provided a means of preventing the hook and fall block from twisting. Thus the bucket always faced the same way without spinning.

At the rear of the main carriage a carrier horn was provided (Fig. 4) to support the carriers, which in turn supported the operating ropes when the carriage was away from the head tower. The auxiliary carriage was hung from the track wheels on hangers so that the sheaves could be swung backward and forward and locked into position to permit adjustment which would change the position of the resultant load and allow better equalization on the track wheels. Lengthening the 1-in. tie between the carriages, as might be necessary in deep canyons, would require this adjustment.

Towers.—The head tower, shown in Fig. 5, was of heavy structural steel, 75 ft 0 in. high supported at the four corners by spherical bearings mounted on 8-wheel trucks designed to equalize wheel loads (Fig. 6). The horizontal thrust was taken by two 4-wheel trucks placed at the rear corners of the tower shown in the foreground of Fig. 6. These wheels were 24 in. in diameter. The floor framing was designed to carry the machinery loads, comprising the hoist, hoist motor, traversing motor, air compressor, and electrical equipment. The side framing on the lower panel consisted of a steel truss designed to carry the floor loads, the bottom chord extending beyond the back of the tower to form a bracket for supporting the thrust wheels and to provide a platform for the counterweight (Fig. 6).

It will be noted that the entire lower part of this tower was covered with corrugated iron housing. The top supported the main pin holding the track cable, and also the sheaves for the endless, hoist, and button cables. Near the top was a platform for servicing the carriage and the various sheaves. It extended 25 ft each side of the center line and, from its ends, messenger cables, on which 1,000-watt electric lights were strung for job illumination, spanned the valley. The control house, about halfway up the tower, was high enough for the operator to have a clear view of the entire work with the possible exception of part of the loading platform just below the near bank.

The tail tower (Fig. 1) was of similar construction except that the floor loads were much lighter. It was 110 ft 0 in. high. The only machinery in this tower was the traversing motor and winch with its attendant electrical equipment.

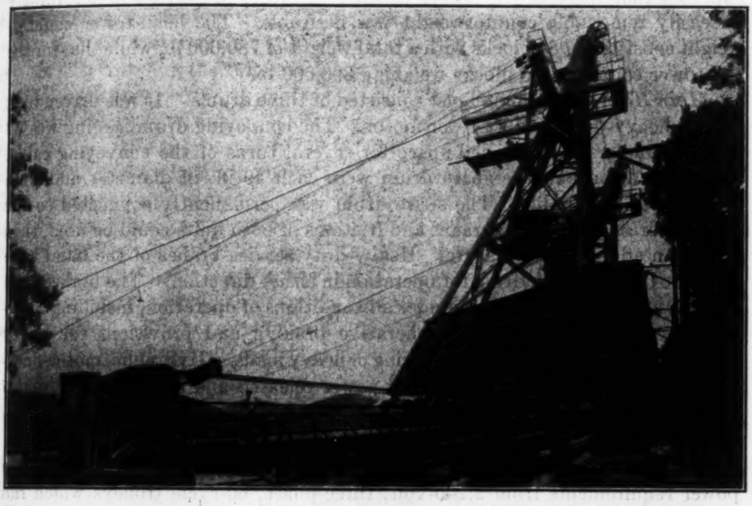


FIG. 5.—GENERAL VIEW OF CABLEWAY HEAD TOWERS, THRUST RAILS, TRUCKS, AND TRAVERSING CABLE

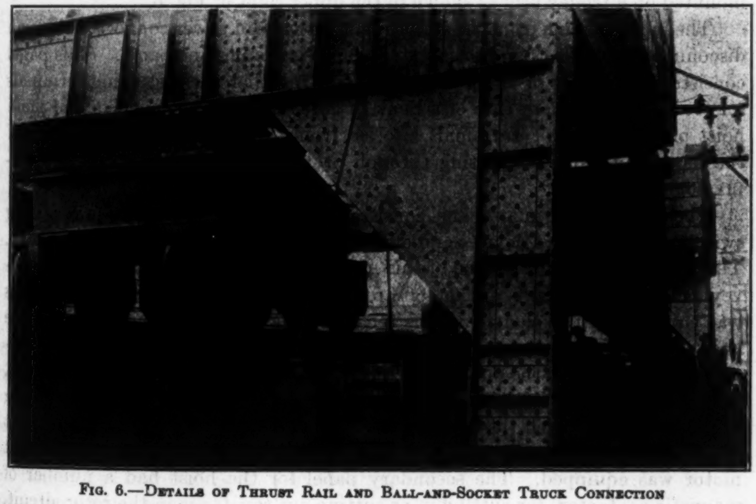


FIG. 6.—DETAILS OF THRUST RAIL AND BALL-AND-SOCKET TRUCK CONNECTION

Counterweights.—Counterweights on these towers, shown in the photographs, rested on two 36-in., 192-lb beams, supported on a bracket made by an extension of the bottom chord of the truss, located at the rear of the towers. These counterweights differed from some installations because they were made up of individual concrete blocks instead of solid blocks poured in place. It was expected that these towers would be used on future projects, and accordingly an easily removable counterweight was desirable. The head-tower counterweight consisted of 84 blocks with a total weight of 780,000 lb, while that for the tail tower contained 96 blocks weighing 890,000 lb.

Main Hoist.—The main hoist consisted of three drums. It was driven by a 400-hp heavy-duty, wound-rotor motor. The conveying drum lagging was 52 in. in diameter and provided space for several turns of the conveying cable. The hoist drum and the spare drum were each 49 in. in diameter and were grooved for $\frac{7}{8}$-in. cable. The spare drum was permanently connected to the hoisting drum so that the brakes and frictions of both units could be used, thus giving an added safety feature. Heavy-duty service brakes of the band type were provided on each drum for operation in either direction. The brakes were capable of sustaining the load under all conditions of operation, including interruption of power supply. Regenerative braking and provisions for motor plugging served to restrain the lowering of heavy loads. Hydraulic, motor-operated, emergency brakes, which were power released and spring set, were also provided on the hoisting motor for operation in case of power failure or overspeed.

Electrical Equipment.—Both head and tail towers secured their respective power requirements from 2,200-volt, three-phase, 60-cycle trolleys which ran parallel to and at the rear of each runway. Suitable trolley collectors were mounted on a trolley arm as shown in the upper right of Fig. 6.

The incoming line from the trolley arm of the head tower connected through disconnecting switches and terminated on the incoming line panel. This panel consisted essentially of an oil circuit breaker together with outgoing terminals for feeding the primary side of a bank of three 25-kva transformers and main hoist primary panel. Terminals also led from the 440-volt secondary side of the bank and, after connecting through protective devices, fed two air-compressor motors.

The main-hoist primary panel consisted of a magnetically-operated reversing switch with protective devices and outgoing terminals for the main-hoist motor. The maximum running torque of this motor which drove the hoisting and conveying drums was 400% of the 400-hp rating. Each of the three hoist drums was equipped with solenoid valves for operating the brake and clutch. The hoist control was so arranged that under normal operation, with the motor connected electrically to the power lines and the drums clutched to the motor shaft, the speed of the motor would be automatically reduced by regenerative braking if the operator allowed the lowering speed of the load to exceed a certain limit as determined by the setting of the overspeed switch with which the hoist motor was equipped. The secondary panel for the hoist had a number of magnetic contactors for cutting in or cutting out resistance in the rotor circuit. Rectifiers provided direct current for the coils of the solenoid valves and the accelerating relays of hoist and travel control.

The traversing motor was similar to the main hoist but operated at 75-hp, 900 rpm, 440 volts, with a maximum running torque of 400% of the 75-hp rating. It was controlled by the traversing control panel which consisted of a magnetic reversing switch, with protective devices and contactors. Power was supplied from the 75-kva transformer bank. The traversing motor was provided with a thruster brake, and when the motor was de-energized the brake set, thereby bringing the tower to a standstill.

Master controllers were provided for both head and tail towers in the control room of the head tower, thereby giving the operator complete control of both towers. Limit switches regulated travel of the tower in each direction, and prevented skew operation between the head and tail towers beyond a certain limit. The tripping of one of these switches resulted in disconnecting power from the traversing motor and the application of its brake. A separate transformer supplied power for lighting in the head tower.

The electrical equipment in the tail tower was similar to that of the head tower, except that only the incoming line panel and traversing control panel were provided. Either tower could be moved at the will of the operator in the head tower. An overhead control cable connected the head and tail towers.

OPERATION AND MAINTENANCE

Method of Operation.—The system of operation of these cableways is commonly known as "full-view operation," as distinguished from "blind operation." The operator in the control cab (Fig. 1) was able to see all the work and observe the performance of the cableway in handling its load. He also had the further advantage of seeing the tail tower. All operations were by remote control from the cab on the head tower.

Instructions from two signalmen were received by telephone through a loudspeaker in the cab. One signalman was at the loading point to control the loading and dispatching operations and the other at the unloading point. All three men were in two-way communication at all times. Every operation was ordered by a signalman and the operator took matters in hand only in emergencies. The value of the visual control system in preventing serious accidents was demonstrated several times on this job.

Mixed concrete was brought to the cableways in transfer cars, running on a standard-gage track extending the full traverse of the cableways, and thence was dumped into cableway buckets resting on a loading platform which extended the full length of the transfer tracks. This arrangement permitted the buckets to be filled at any point along the cableway traverse. Each car, carrying 6 cu yd, required from 18 to 30 sec to dump, depending upon the slump of the concrete.

For delivering concrete a bucket of special TVA design (Fig. 7) was carried on the hook continuously. It was a bottom-dump type, weighing about 7,000 lb. Its capacity was 7½ cu yd, and normally it carried 6 cu yd of concrete. Particularly noteworthy is the fact that the entire bottom of this bucket drops away for quick emptying.

To cause the filled bucket to swing clear of the transfer tracks the carriage was run a few feet toward the tail tower just before hoisting. The bucket was then hoisted high enough to clear all obstructions and the carriage was run out to a spot over the point of deposit. There the carriage was brought to a stop so as to eliminate swing of the bucket. After lowering and dumping, the bucket was again raised to clear obstructions and the carriage returned to the head tower. This cycle could be completed to any spot within range of the cableway within 4 min. At close range, it was possible to effect a 2-min. cycle. Operation of the two cableways, 3 transfer trains, and 3 mixers were coordinated by a dispatcher stationed in the mixer plant, making it possible to supply concrete of different mixes at different points at the same time with very little difficulty.

Fig. 7.—Air-Dump 6-Yd Bucket Developed at Norris Dam

During peak production, four crews worked five and one half hours each, although the cableways operated twenty-three hours per day, one hour being taken at noon for inspection and greasing. In addition to the general cableway foreman, the normal operating crew necessary for two cableways was as follows: Two operators each shift, one head-tower oiler each shift, one tail-tower oiler each shift, four signalmen each shift, one foreman 8 hr per day, and one foreman 6 hr per day. The foremen acted as spare operators when necessary.

Most of the operators had had previous cableway experience before their employment at Norris Dam. The signalmen and the oiler in the head tower were riggers who showed special ability at their respective jobs, and the tail-tower oiler was a semi-skilled laborer. Each of the foremen was a former operator.

At first the 6-yd square buckets were not satisfactory because several men were needed to dump them. Air operation of the dump was devised because of the speed and saftey, and this was used successfully throughout the job. Considerable study was made to lessen the "bounce" of the bucket when it was dumped. Finally an inverted V-shaped baffle was installed across the center of the bucket which slowed up the discharge sufficiently to reduce the "bounce" to a minimum.

Cable Maintenance.—The largest recurrent repair item was replacement of the operating cables. Whenever possible, this was undertaken on Sundays, but in many cases changes had to be made during normal working hours. At first these changes took considerable time, but later they could be made in an average of one hour for button or hoist cable and three hours for endless line. Cables were inspected and greased each day during peak operation but later they were inspected every second day and greased once a week. While this maintenance and replacement could be made with four riggers, often more were used to speed up the operation. These men formed a separate rigger crew and were used for all similar operations on the job.

The track cable was turned one eighth of a revolution about its horizontal axis every six weeks to present a new rolling surface to the carriage wheels. At first the button lines failed because of excessive wear just in front of the buttons, apparently caused by the slack carrier rebounding after striking the button and hitting the cable. This was overcome to a considerable extent by slipping a pipe sleeve about 18 in. long over the cable in front of the button.

When both the cableways had operated a total of 16,947 hr the record of cable replacement was as given in Table 2. After the cableways had been operated for a sufficient period, discarded endless and hoist lines were used to replace worn traversing cables.

TABLE 2.—RECORD OF CABLE DURABILITY

Cable	Size and type	Number replaced	Operating hours each	Material cost per operating hour
Button	¾-in. regular lay	10	534.3	$0.591
	¾-in. flattened strand	5	1,586.8	0.286
Endless	1-in. regular lay†	7	1,149.3	0.795
	1-in. flattened strand	4	1,467.6	0.888
Hoisting	⅞-in. regular lay	16	887.8	0.519
	⅞-in. flattened strand	1	806.5	0.851
Traversing (new only)	⅞-in. 6 by 19 regular	6	1,936.0	0.333
Take-up	1⅜ in. 6 by 19 regular	2	5,625.0	0.053
Endless drum wearing plates*	With regular lay	6	567.6	0.577
	With flattened strand‡	2	2,478.6	0.129

* Partial record. † Does not include one cable that broke after 86 hr of service. ‡ Note the increased life of the wearing plates with the use of flattened strand.

TABLE 3.—QUANTITIES OF MATERIAL, OTHER THAN CONCRETE, HANDLED BY CABLEWAYS

Item No.	Material	In tons
1	Construction machinery	210
2	Dam closure operation	223
3	Dismantling concrete, No. 3 cofferdam	3,850
4	Drum-gate steel and machinery	680
5	Excavation	22,500
6	Form hoists	11,250
7	Form lumber	3,400
8	Grout cement	4,840
9	Penstock steel	586
10	Pipe, conduit, etc.	500
11	Power house permanent equipment	1,760
12	Power house structural steel	810
13	Reinforcing steel	2,800
14	Riprap, east embankment	8,190
15	Road material and maintenance	875
16	Spillway bridge steel	375
17	Spillway tube gates, castings, and operating equipment	1,087
18	Tile and rock for drains	900
19	Timber cribs for cofferdams	4,375
20	Tractor gates and operating machinery	385
21	Trashrack steel	575

Performance.—Until April, 1936, which was practically at the end of concreting operations, concrete had constituted about 2,000,000 tons or 96.96% of the tonnage moved by cableway. The estimated tonnage of other equipment and materials handled is as given in Table 3.

Material which came in by truck was unloaded at a road which extended to the base of the dam next to the power house on the left bank of the river. For such unloading the cableways traveled downstream from the dam. Since the carpenter shop was located within the range of the cableways, forms built there could be moved directly to the point at which they were to be used.

The maximum placement of concrete in a single month was 92,780 cu yd for both cableways in April, 1935. In this period the average hourly placement was 157.3 cu yd. The maximum 24-hr placement was made on January 7, 1935, when 4,090 cu yd were placed for an average of 170.4 cu yd per hr. The maximum rate in a single shift was made July 10, 1935, when 1,445.8 cu yd were poured between 4 p.m. and 12 p.m. for an average of 180.7 cu yd per hr. Under perfect conditions one cableway placed 162 cu yd per hr, but normally the continuous rate was limited to approximately 120 yd per hr.

Three 3-cu yd concrete mixers, each operating on a 3-min cycle, limited concrete placement to 180 cu yd per hr. It was the practice to stagger the time required for handling other material such as lumber, reinforcing steel, or forms by the cableway throughout a shift, because one cableway could be spared for short periods and return to clear the accumulated concrete from the loading track without delaying the mixing plant. The same condition was encountered in changing from one form to another. One cableway carried more than its share of the concrete until the crew which had changed to another form was ready to begin placing at the new location.

In summary form, Table 4 indicates the quantities of concrete and other materials handled. As noted, delays amounted to 9.3% and 10.9% of the gross operating time for cableways Nos. 1 and 2, respectively. During the period covered by Table 4 each cableway consumed an average of 66.2 kw of electricity per gross operating hour, exclusive of lighting.

Dual Use of Cableways.—Although each cableway operated separately under ordinary conditions, both were used for placing loads in excess of the capacity of one cableway but within the combined capacity of both. This arrangement was used to place the following material:

Material	Weight, in tons
16 penstock sections, 20 ft in diameter by 20 ft long	36–40
2 power house crane girders, 62 ft 3 in. long	33
1 generator rotor	35
8 generator stator frame sections	30
2 turbine water wheels	51
6 spillway bridge girders, 107 ft long, 7 ft 1 in. deep	40

As an example of how heavy loads were handled by two cableways, Fig. 8 shows a 40-ton bridge girder being carried from a trailer at the concrete-loading platform to its final position over the spillway. For this work a 36-in., 175-lb lifting beam, 29 ft 6½ in. long, weighing 6,800 lb, was used. At each end 6-in. by 1-in. bars were fastened to engage the shackles from the cableway fall blocks. At the center a 4-in. eye-bolt, with a nut swiveling on a roller bearing, turned in a bolster which was clamped to the beam with four 2-in. rods. In order to strengthen the beam against lateral deflection, four 1-in. rods trussed the top and bottom flanges.

TABLE 4.—NORRIS DAM CABLEWAY PERFORMANCE JULY 17, 1934, TO APRIL 1, 1936

Description	Cableway No. 1	Cableway No. 2	Total
(*a*) TIME OPERATED, IN HOURS			
Elapsed	14,496	14,399	28,895
Idle	3,713	4,243	7,956
Gross operating	10,783	10,156	20,939
Delays			
Mechanical	285	374	659
Cable replacement	147	120	267
Waiting	390	438	828
Maintenance	176	175	351
Total	998	1,107	2,105
Net operating	9,785	9,049	18,834*
Placing Concrete			
Dam	6,774	5,451	12,225
Power house	230	387	617
Core wall	229	154	383
Miscellaneous	62	93	155
Total	7,295	6,085	13,380
Total other work	2,490	2,964	5,454
(*b*) QUANTITIES HANDLED			
Concrete, in Cubic Yards			
Dam	555,460	408,011	963,471
Power house	12,485	11,482	23,967
Core wall	8,292	2,845	11,137
Miscellaneous	1,603	4,752	6,355
Total concrete	577,840	427,090	1,004,930
Other work, in tons	35,010	33,272	68,282
Concrete in Cubic Yards per Net Hour			
Dam	82.0	74.9	156.9
Power house	54.3	29.7	84.0
Core wall	36.2	18.5	54.7
Miscellaneous	25.9	51.1	77.0
Average for all concrete	79.2	70.2	149.4
Other work in tons per net hour	14.1	11.2	25.3

* The total net operating hours at the close of the job was 20,918.

A 30-in. pendulum, with a target, was attached near the center of the beam to assist signalmen in keeping the beam level. One signalman controlled both cableways and it was necessary to plug in his telephone at three different points during the travel of the load. About four hours were required to transfer the first girder because some time was lost in checking the sag of the main cables and in maneuvering the trailer on the loading platform. The remaining girders required approximately two hours each to place.

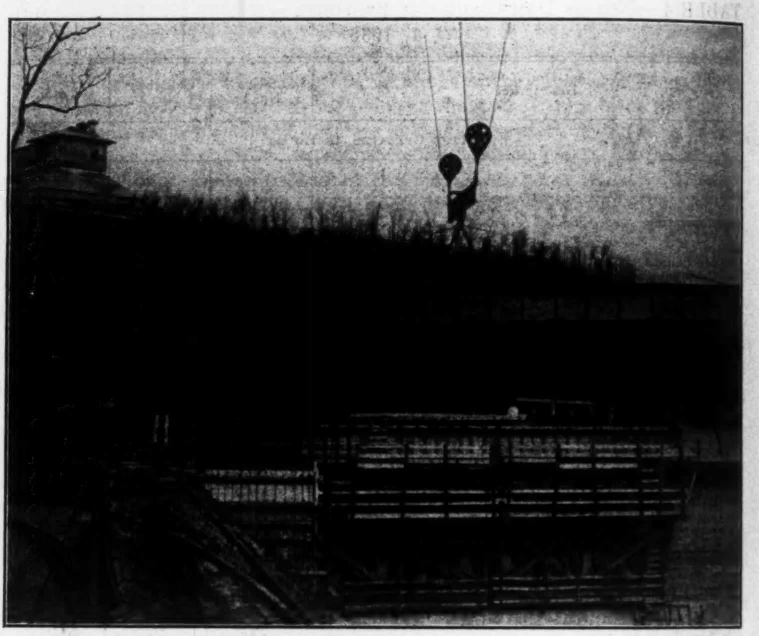
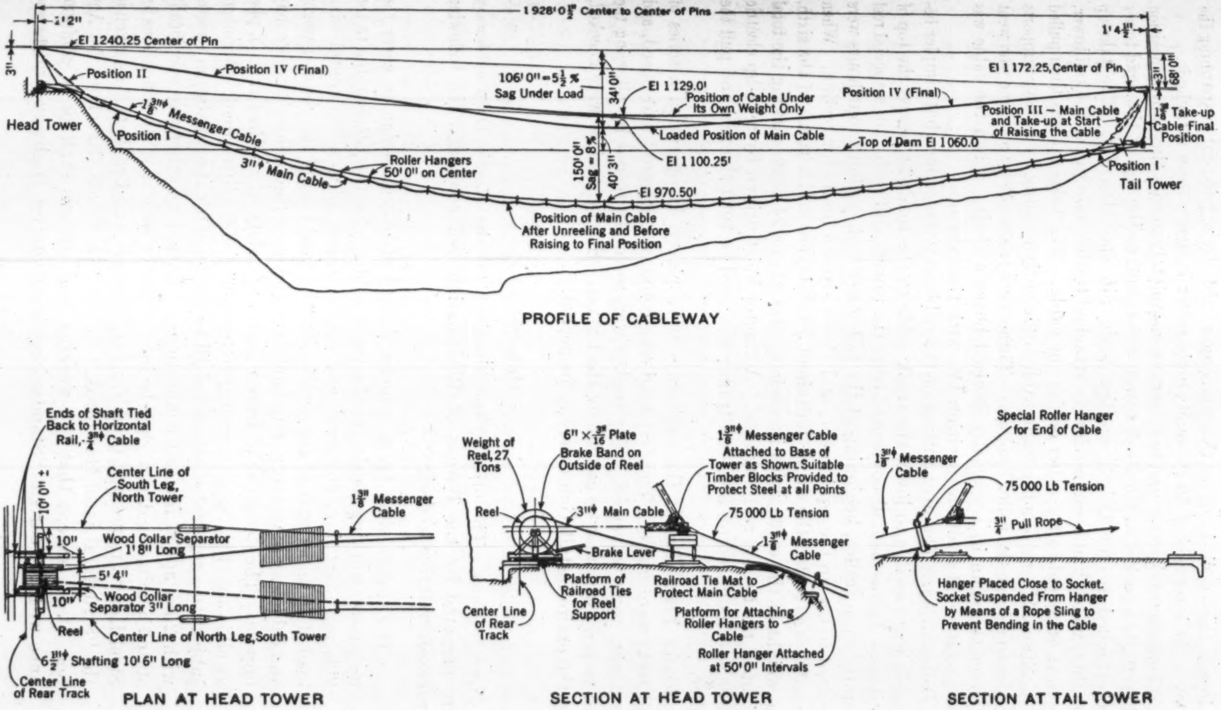


FIG. 8.—TYPE OF HEAVY LOAD HANDLED BY BOTH CABLEWAYS WORKING AS A UNIT. FORTY-TON GIRDER FOR SPILLWAY BRIDGE IS BEING PLACED

The turbine water wheels constituted the heaviest units handled by the cableways. This load was lifted about 1 ft and carried about 18 ft utilizing the lifting beam just described. The 48-ton wheel together with the 6,800-lb lifting beam made a total load of 51 tons, or 25½ tons per cableway.

TRACK CABLE ERECTION AND DISMANTLING

Erection.—The track cable was erected according to the plan and sequences shown in Fig. 9. A 1⅜-in. messenger cable was strung across the valley with its ends fastened to the lower front corner of the towers. The cable reel was set up on a temporary support located between the two head towers. An improvised brake band was placed on two flanges of the cable reel. The cable was first unreeled to bring the socket to the edge of the cliff where a special hanger was attached. This hanger was approximately 6 ft long; it consisted of a single roller on the 1⅜-in. messenger cable with an attachment at the other end for clamping to the cable socket, to prevent the cable from rotating. The track



PROFILE OF CABLEWAY

PLAN AT HEAD TOWER

SECTION AT HEAD TOWER

SECTION AT TAIL TOWER

1 928' 0½" Center to Center of Pins
1' 2"
El 1240.25 Center of Pin
3"
1' 4½"
3"
68' 0"
Position II
Position IV (Final)
106' 0" = 5½ % Sag Under Load
34' 0"
El 1 129.0'
Position of Cable Under Its Own Weight Only
El 1 172.25, Center of Pin
Position IV (Final)
Position III – Main Cable and Take-up at Start of Raising the Cable
1⅜" Take-up Cable Final Position
Head Tower
Position I
1⅜"ϕ Messenger Cable
Loaded Position of Main Cable
Top of Dam El 1 060.0
El 1 100.25'
150' 0" Sag = 8%
40' 3"
3"ϕ Main Cable
Roller Hangers 50' 0" on Center
El 970.50'
Position I
Tail Tower
Position of Main Cable After Unreeling and Before Raising to Final Position
Ends of Shaft Tied Back to Horizontal Rail, ¾"ϕ Cable
Center Line of South Leg, North Tower
10' 0" ±
1⅜" Messenger Cable
10"
Wood Collar Separator 1' 8" Long
5' 4"
10"
Wood Collar Separator 3" Long
Reel
Center Line of North Leg, South Tower
6 1/2"ϕ Shafting 10' 6" Long
Center Line of Rear Track
Weight of Reel, 27 Tons
6" × 3/16" Plate Brake Band on Outside of Reel
1⅜"ϕ Messenger Cable Attached to Base of Tower as Shown. Suitable Timber Blocks Provided to Protect Steel at all Points
Reel
3"ϕ Main Cable
75 000 Lb Tension
Brake Lever
1⅜"ϕ Messenger Cable
Platform of Railroad Ties for Reel Support
Railroad Tie Mat to Protect Main Cable
Center Line of Rear Track
Platform for Attaching Roller Hangers to Cable
Roller Hanger Attached at 50' 0" Intervals
Special Roller Hanger for End of Cable
1⅜"ϕ Messenger Cable
75 000 Lb Tension
¾" Pull Rope
Hanger Placed Close to Socket. Socket Suspended From Hanger by Means of a Rope Sling to Prevent Bending in the Cable

FIG. 9.—ERECTION PLAN FOR MAIN CABLE

cable was then pulled out on the messenger cable by a ¾-in. cable spanning the valley. At intervals of 50 ft small single-roller hangers were attached.

When the track cable had been strung completely across the valley (position I, Fig. 9), the socket at the head tower was raised to the top of the head tower and attached to the U-bolt on the main pin (position II). The take-up assembly, which had been previously attached to the main pin of the tail tower, was next attached to the other end of the cable. The take-up was then pulled up (position III), raising the track cable sufficiently to hang by its own supports in order to release the roller hangers. These were taken off by a rigger who went out over the span in a specially devised bosun's chair. Then the cable was raised to its final position (position IV) with the take-up.

Dismantling.—The dismantling of the cableway was somewhat simpler because it was possible to lower the track cable to the roadway across the top of the dam. It was then disconnected from the towers and reeled on a special reel mounted on a trailer, beginning at the tail-tower end. About four hours were required to set up the equipment, and about sixteen hours for reeling. When the cable was reeled the drum contained 5 full layers and 2 turns of the sixth. The weight of the cable, with its sockets, was about 24 tons, making the total weight of the loaded reel 26½ tons. A tractor hoist driven by a 20-hp electric motor and mounted on a 40-ton trailer was used to turn the reel and pull the trailer.

Service Test.—Before the cableways were placed in operation a series of load tests were made. The test load was conveyed, lowered and raised, and the towers were traversed. The brakes were severely tested by allowing the load to run away and be caught by the brakes. Safety devices were checked, and the entire installation carefully inspected.

Design

Specifications.—The original specifications for the Norris Dam cableways were prepared by the Bureau of Reclamation and contained the following paragraphs relative to design:

> The towers shall be of structural steel of liberal design to carry the maximum working tension of the track cables, plus the tension due to the hoisting, dumping, conveyer and carrier ropes, plus a horizontal wind load of 30 lb per sq ft of gross area, plus the dead load of the counterweight and cableway machinery, plus such other loads as the builder's design may impose. The design shall have a factor of safety of not less than 5, plus an allowance for impact stresses produced by ¼ of the lifted load. Members subject to a reversal of stresses shall be designed for the maximum stresses with 25% added for end connections. Except as otherwise specifically stated herein, the design shall be in accordance with the "Specifications for Steel Head Frames and Coal Tipples, Washers and Breakers," as given in the "Structural Steel Handbook," by the late Milo S. Ketchum, M. Am. Soc. C. E., except that the working load stresses shall be in conformance with the current "Standard Specifications for Highway Bridges and

> Incidental Structures" of the American Association of State Highway Officials.
>
> The carriages and fall blocks shall be designed for maximum hook loads of 18 tons on either hoisting or dump blocks, with a factor of safety of not less than 5. The carriages shall be made in two sections to permit spreading of fall lines when operating from the maximum hoisting distance. The track rollers shall be not less than 24 in. in diameter with machined grooves having a depth of not less than three quarters of the rope diameter. Each roller shall develop a load of not to exceed 5,000 lb on the track cable. Proper provision shall be made for equalization to avoid excessive loading at any point.
>
> The size of track cables shall be based on a factor of safety of 3, as shall be determined by test, which shall take into consideration the maximum hook load, weight of carriage and fall blocks, ropes, horizontal component due to tension, and any other factors that may produce stress.

Subsequently, an addendum was issued by the TVA in which the following statement was added:

> The weight of the counterweight shall be determined by increasing by 25% the balancing force required under the condition of lifting 25 tons with a 5½% sag on the track cable.

The specifications called for a span of 1,898 ft with a track cable deflection under load of 5½%. Later, the span was lengthened to 1,925.5 ft.

Conditions Used in the Design Check.—The towers were checked for the conditions of loading and stress given in Table 5. The impact is defined as the additional stress produced by the effect of vertical oscillation of the track cable. This is most pronounced when a concrete bucket is dumped or when a bucket which is being raised or lowered is stopped suddenly.

The Lidgerwood Manufacturing Company, American Steel and Wire Company, and the Virginia Bridge Company collaborated in the design of the towers and equipment, and the resulting design was checked by the Construction Plant Division of the TVA. Wind load on exposed areas did not control except in certain bracing members. Unless this stress amounted to more than 25% of the combined dead-load and live-load stresses it was neglected.

For the loads listed in Table 5 the entire horizontal component was considered acting at the base of the tower through the horizontal thrust wheels. The design of the towers presented no special structural problem after the loads were computed. In checking the design the condition with load at midspan gave the maximum stress in the cable and therefore in the towers. Stresses in the track cable and operating cables were also computed with the load at the loading point, and at 140 ft from the tail tower. These two conditions gave maximum stresses in the outhaul and inhaul cables but did not affect the tower design.

Track Cable Stress.—In calculating the stress in the track cable, the formulas adopted were those ordinarily used, as follows:

TABLE 5.—DESIGN FACTORS FOR TOWERS

Loadings	Items	Allowable Stresses
(a) CASE 1		
Dead load	Tower, machinery, counterweight	Tension: 13,000 lb per sq in.
Track cable pull (5½% sag)	Weight of cable, carriage, hoist, blocks and bucket plus 18-ton live load	Compression: $\dfrac{13{,}000}{1 + \dfrac{L^2}{13{,}500\, r^2}}$
Operating cable pulls	Conveyer, button, hoist	
Light cable pull	2 cables at 30,000 lb each	Factor of safety = 5
(b) CASE 2		
Dead load	Tower, machinery, counterweight	Tension: 16,000 lb per sq in.
Track cable pull (5½% sag)	Same as Case 1 adding 25% impact to 18-ton live load	Compression: $\dfrac{16{,}000}{1 + \dfrac{L^2}{13{,}500\, r^2}}$
Operating cable pulls	Conveyer, button, hoist	
Light cable pull	2 cables at 30,000 lb each	
Wind	30 lb per sq ft on exposed area	Factor of safety = 4
(c) CASE 3 (CABLES OFF)		
Dead load	Tower, machinery, counterweight	Tension: 16,000 lb per sq in. Compression: $\dfrac{16{,}000}{1 + \dfrac{L^2}{13{,}500\, r^2}}$ Factor of safety = 4
(d) CASE 4 (EXTREME CONDITION)		
Dead load	Tower, machinery, counterweight	For this condition 10% increase in stress over Case 2 allowed
Track cable pull (5½% sag)	Same as Case 1 with 25-ton live load	
Operating cable pulls	Conveyer, button, hoist	
Light cable pull	2 cables at 30,000 lb each	

NOTES.—

Maximum $\frac{L}{r}$
- 100 for main compression members
- 140 for secondary compression members
- 200 for secondary compression members

Allowable Stresses
- Shear on rivets = 10,000 lb per sq in.
- Bearing on rivets = 20,000 lb per sq in.
- Shear on beam webs = 8,100 lb per sq in.

For load at midspan (see Fig. 10(a)):

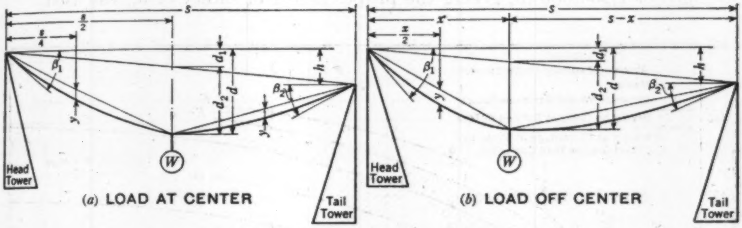
$$d = (d_1 + d_2) = \frac{h}{2} + 0.055\, s \ldots\ldots\ldots\ldots\ldots (1a)$$

$$t = \frac{s\,(2\,W + w\,s)}{8\,d - 4\,h} \ldots\ldots\ldots\ldots\ldots (1b)$$

$$y = \frac{w\left(\frac{s}{2}\right)^2}{8\,t} \ldots\ldots\ldots\ldots\ldots (1c)$$

$$\tan \beta_1 = \frac{\frac{h}{4} + \frac{d_2}{2} + 2\,y}{\frac{s}{4}}\,; \qquad \tan \beta_2 = \frac{\frac{d_2}{2} - \frac{h}{4} + 2\,y}{\frac{s}{4}} \quad \ldots\ldots\ldots (1d)$$

$$T_1 = \frac{t}{\cos \beta_1}\,; \qquad T_2 = \frac{t}{\cos \beta_2} \quad \ldots\ldots\ldots (1e)$$



(a) LOAD AT CENTER (b) LOAD OFF CENTER

FIG. 10.—DESIGN NOMENCLATURE

For load at points other than midspan (see Fig. 10(b)):

$$d = (d_1 + d_2) = \frac{x\,h}{s} + \frac{x\,(s - x)\,(w\,s + 2\,W)^2}{2\,t\,[w\,s^2 + 4\,W\,\sqrt{x\,(s - x)}]} \quad \ldots\ldots (2a)$$

$$t_1 = \frac{w\,x\,(s - x)}{2\,d_2} + \frac{W\,x\,(s - x)}{s\,d_2} \quad \ldots\ldots\ldots (2b)$$

$$y = \frac{w\,x^2}{8\,t_1} \quad \ldots\ldots\ldots (2c)$$

$$\tan \beta_1 = \frac{d + 4\,y}{x} \quad \ldots\ldots\ldots (2d)$$

$$T_1 = \frac{t_1}{\cos \beta_1} \quad \ldots\ldots\ldots (2e)$$

The following nomenclature and data were used, in addition to those shown in Fig. 10:

W = total concentrated load in pounds (58,000 lb including carriage, fall block, cables, etc.);
w = weight of cable in pounds per foot = 22.2 lb;
s = span in feet = 1,925.5 ft;
h = difference in elevation of supports in feet = 68 ft;
d = total deflection at load point from highest support in feet;
x = distance from highest support to the load in feet;
t = horizontal component of cable tension in pounds with load at center;
t_1 = horizontal component of cable tension in pounds with load at a point other than center;
T_1 = cable tension at high tower in pounds;
T_2 = cable tension at low tower in pounds;
y = deflection at center of one-half span or part span in feet;
β_1 = angle cable makes with the horizontal at high support;
β_2 = angle cable makes with the horizontal at low support.

Curves showing the computed maximum stress for a 17-ton load, with and without impact as computed by these formulas, are shown in Fig. 11 (curves *A*, *B*, and *C*). Original computations gave a stress of 362,000 lb with an 18-ton load at midspan without impact. The curves are drawn for a 17-ton load in order to be comparable to the stress tests which were made later under an actual load of 17 tons. A 3-in. locked coil cable which has a tested breaking strength of 1,100,000 lb, giving the proper factor of safety of 3, was used.

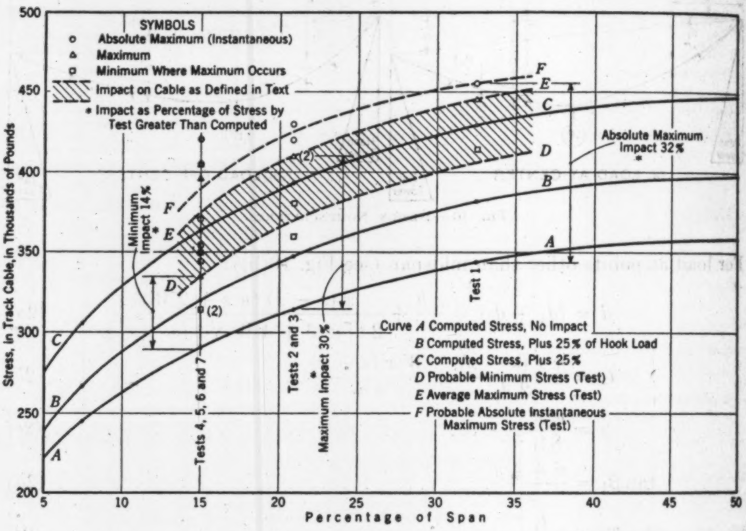


FIG. 11.—COMPARISON OF COMPUTED AND OBSERVED STRESSES IN MAIN CABLE

Carriage and Operating Cables.—The stresses in the conveyer cable were determined by a graphical analysis of the carriage in order to determine the variation in the wheel loads and also to check the conveyer cable pull. The carriage was assumed to be in a rest position at three points—midspan, loading point, and 140 ft from the tail tower. With the carriage at rest the resultant is perpendicular to the track cable. The result of this computation was a carefully designed carriage which fixed the position of the hoist sheaves in both the auxiliary and main carriages in such positions that the resultants passed as nearly as possible through the centers of the carriages. It is not possible to maintain this condition throughout every position on the cableway span, but it was possible to keep the individual wheel loads within the specified 5,000 lb. A series of computations for the pull of operating cables gave the data shown in Table 6.

Light and Conductor Cables.—For the tension from the light cables and conductor cables it was assumed that there was a $5\frac{1}{2}\%$ sag under their own dead weight. The extent and number of light cables was not known definitely at first. Due to this uncertainty the total pull of light and conductor cables was assumed at 60,000 lb. A final check reduced this to 32,300 lb.

TABLE 6—MAXIMUM PULL ON TOWERS FROM OPERATING CABLES (OCCURS WHEN LOAD IS IN CENTER)

Cable	LOAD, IN POUNDS		
	At loading point	At midspan	140-ft from tail tower
(a) HEAD TOWER			
Button	6,000	6,000	6,000
Outhaul	13,000	13,000	19,000
Inhaul	14,500	1,200	0
Hoist	9,900	9,900	9,900
Dump*	970	970	970
Total	44,370	31,070	35,870
(b) TAIL TOWER			
Button	6,000	6,000	6,000
Outhaul	13,000 13,000	13,000 13,000	19,000 19,000
Total	32,000	32,000	44,000

* Later omitted although included in computations.

Counterweight.—The counterweight was calculated exactly as detailed in the specifications. It equaled 25% more than a weight sufficient to balance the tower with a 25-ton load at midspan.

Wheel Loads and Reactions.—A summary of the tower reactions and wheel loads is given in Table 7. The maximum wheel load occurs when all the cables are off. This amounts to 77,500 lb. However, this condition never occurred because the full counterweight was not placed on the tower until after the main cable was in place. Actually the maximum wheel load, amounting to about 66,000 lb, probably occurred when the cableway was lifting a 25-ton load. The vertical wheels were crane type, rolled steel, 27 in. in diameter.

Temperature Effect.—The effect of temperature on the stress of the main cable was carefully checked after the cableway was in operation. The sag of the cable was checked for the unloaded cable at various temperatures, and a curve showing sag against temperature was drawn (Fig. 12). A corresponding curve showing the sag of the loaded cable plotted against temperature was calculated, as shown in Fig. 12. The stress in the cable for both loaded and unloaded conditions was then plotted over a range of 100° F. It will be seen that the change in stress due to 100° F was the difference between 340,000 lb and 375,000 lb, which is approximately 10%. With a factor of safety of 3 for the cable and the liberal factor of safety of 5 on the towers, this variation due to temperature could safely be neglected provided the sag of the cable was maintained within the limits of the contract, namely, 0.055 of the span plus or minus 5%. In summer the cable had to be raised so that the correct sag occurred at about 75° instead of at 33°.

TABLE 7.—LOADS AND REACTIONS ON CABLEWAY TOWERS, IN THOUSANDS OF POUNDS, SUMMARIZED FROM ORIGINAL COMPUTATIONS

Loads and reactions	LOADING CONDITIONS				All cables off-dead load of tower and counterweight only
	Dead-weight cable only; no live load	Dead-weight cable and carriage plus 18-ton live load, at center; 5½% sag	Dead-weight cable and carriage plus 18-ton live load plus 25% impact†; 5½% sag	Dead-weight cable and carriage plus 25-ton live load, at center; 5½% sag	
Head Tower:					
Main cable pulls*	163	396	446	460	....
Light and conductor cable pull	....	60	60	60	....
Counterweight	780	780	780	780	780
Tower, trucks, and machinery	453	453	453	453	453
Total Tower Reactions:					
Front track vertical	348	932	1,020	1,058	12
Back track vertical	895	360	280	245	1,203
Horizontal	161	451	500	514	....
Wheel Loads:					
Front vertical	22	58.2	64	66	0.750
Back vertical	56	22.4	17.5	15.3	75
Horizontal	20.1	56.4	62.5	64.3	....
Tail Tower:					
Main cable pull*	162	393	442	456	....
Light and conductor cable pull	....	60	60	60	....
Counterweight	890	890	890	890	890
Tower, trucks, and machinery	361	361	361	361	361
Total Tower Reactions:					
Front track vertical	329	902	1,000	1,018	−5.8
Back track vertical	930	375	282	250	1,237
Horizontal	160	448	496	510	....
Wheel Loads:					
Front vertical	20.6	56.3	62.5	63.6	−0.360
Back vertical	58.2	23.4	17.6	15.6	77.5
Horizontal	20	56	62	63.7	....

* Includes 31,000 lb pull from operating ropes (button, conveyer, hoist, and dump).

† Impact in these computations was taken as 25% of the 18-ton load.

Traversing Force.—The force necessary to traverse the tower was arbitrarily assumed to be 80,000 lb, which was the expected pull exerted on the anchors at either end of the runway. This is a highly indeterminate figure and depends on the smoothness of the track, friction of the wheel bearings, and positions of various loads on the tower. It also varies with the length of time the tower remains standing still, and is usually considerably higher when starting than when actually traversing. Experience on the Madden cableway dictated the choice of 80,000 lb. Test results, as shown subsequently, indicate that this was a fair value.

Design of Runways.—Track for the vertical wheels was designed to take the heaviest wheel loads, and consisted of the following: 131-lb A.R.E.A. Rail and 7 by 9 in. by 8 ft oak ties 18 in. on center. The rear track was supported on a concrete base designed with a vertical cantilever on the rear side. This

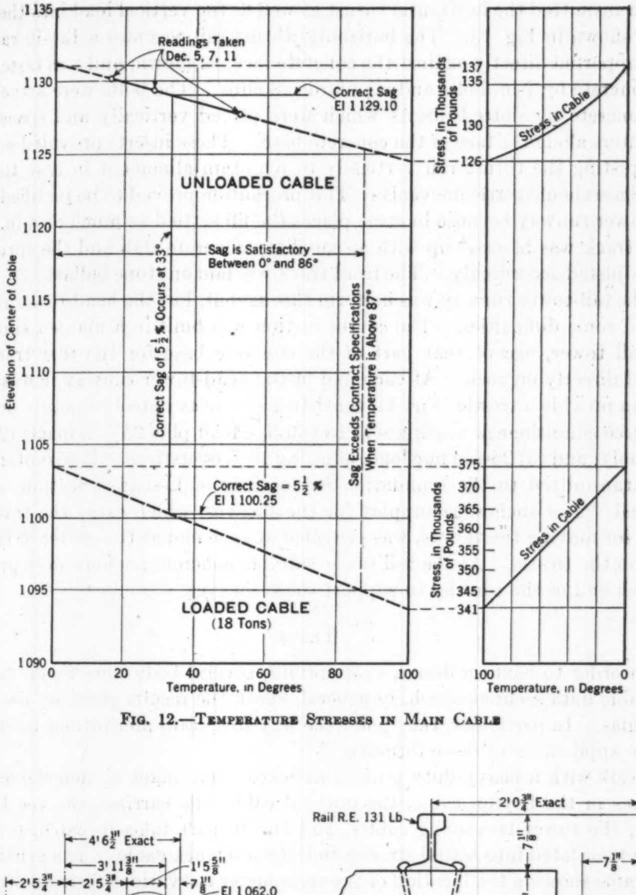
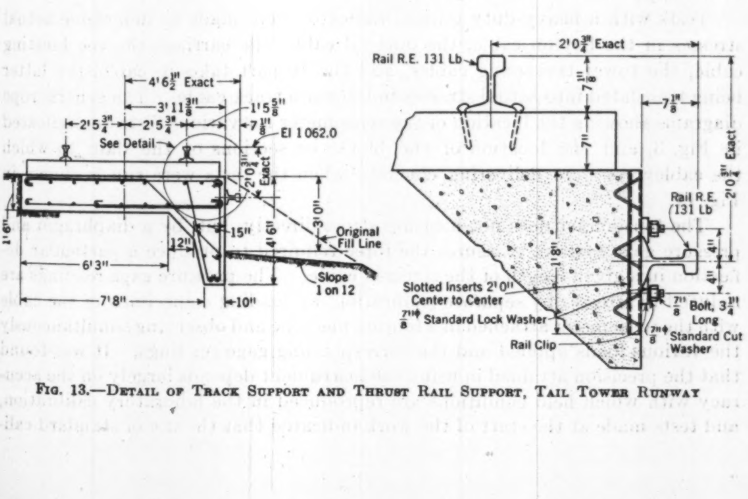


FIG. 12.—TEMPERATURE STRESSES IN MAIN CABLE



4'6½" Exact
3'11⅜"
1'5⅝"
2'5¾"
2'5¾"
7⅛"
El 1 062.0
See Detail
2'0¾" Exact
3'0"
4'6"
15"
12"
1'6"
6'3"
7'8"
10"
Original Fill Line
Slope 1 on 12
2'0¾" Exact
Rail R.E. 131 Lb
7⅛"
7⅛"
7"
2'0¾" Exact
Rail R.E. 131 Lb
1'8"
4"
4"
Slotted Inserts 2'0" Center to Center
⅞"φ Standard Lock Washer
Rail Clip
⅞" Bolt, 3¾" Long
⅞"φ Standard Cut Washer

FIG. 13.—DETAIL OF TRACK SUPPORT AND THRUST RAIL SUPPORT, TAIL TOWER RUNWAY

base transmitted the horizontal thrust as well as the vertical load into the earth fill as shown in Fig. 13. The horizontal thrust rail was also a 131-lb rail. It was supported directly against the concrete face, as shown, and was fastened to the concrete by ⅞-in. bolts and standard rail clips. The bolts were attached to the concrete by slotted inserts which were placed vertically and spaced 2 ft on centers along the face of the concrete base. These inserts provided a means of adjusting the thrust rail vertically to maintain alinement in case the concrete base should settle unevenly. This precaution proved to be justified in the tail-tower runway because in some places the fill settled as much as 6 in. The main track was blocked up with an auxiliary concrete slab and the thrust rail was adjusted accordingly. The front track was laid on stone ballast.

The tail-tower runway was uniform throughout, but the head-tower runway offered some difficulties. The center section was built in a manner similar to the tail tower, except that part of the concrete base for the rear track was placed directly on rock. At each end of the head-tower runway it was necessary to provide a trestle (Fig. 1), consisting of a heavy steel structure designed for three conditions of loading: (1) 18-ton live load plus 25% impact; (2) dead load only; and (3) 25-ton live load, allowing 10% overstress. Horizontal thrust was transmitted to the foundation by footings of L-shaped section, one leg vertical. The anchorage support for the sheaves, which carry the traversing cable for moving the towers, was provided at each end at the center of the runway on the trestle. At the tail tower suitable concrete anchors were provided at each end of the runway to support these sheaves.

TESTS

In order to confirm design assumptions, several tests were made and considerable data secured which, in general, check the results given by the design formulas. In particular, they point the way to certain precautions to be taken in the application of these formulas.

Tests with a heavy-duty tension indicator were made to determine actual stresses in the button cable, the outhaul cable, the carriage tie, the hoisting cable, the tower traversing cables, and the 10-part take-up cable, the latter being translated into actual stresses in the main track cable. The general rope diagrams showing the location of the tensometer for various tests are indicated in Fig. 3, and the location of the blocks or sections of the dam to which the cableways were delivering concrete when the tests were run is shown in Fig. 14.

The tensometer does not measure stress directly, but, by a diaphragm and pressure gage system, measures the force required to produce a particular deflection in a given length of the stressed cable. The pressure gage readings are reduced to stresses by separate calibration, by loading a specimen of the cable with the tensometer attached in a tension machine and observing simultaneously the various loads applied and the corresponding gage readings. It was found that the precision attained in using the instrument depends largely on the accuracy with which field conditions are reproduced in the laboratory calibration, and tests made at the start of the work indicated that the use of standard cali-

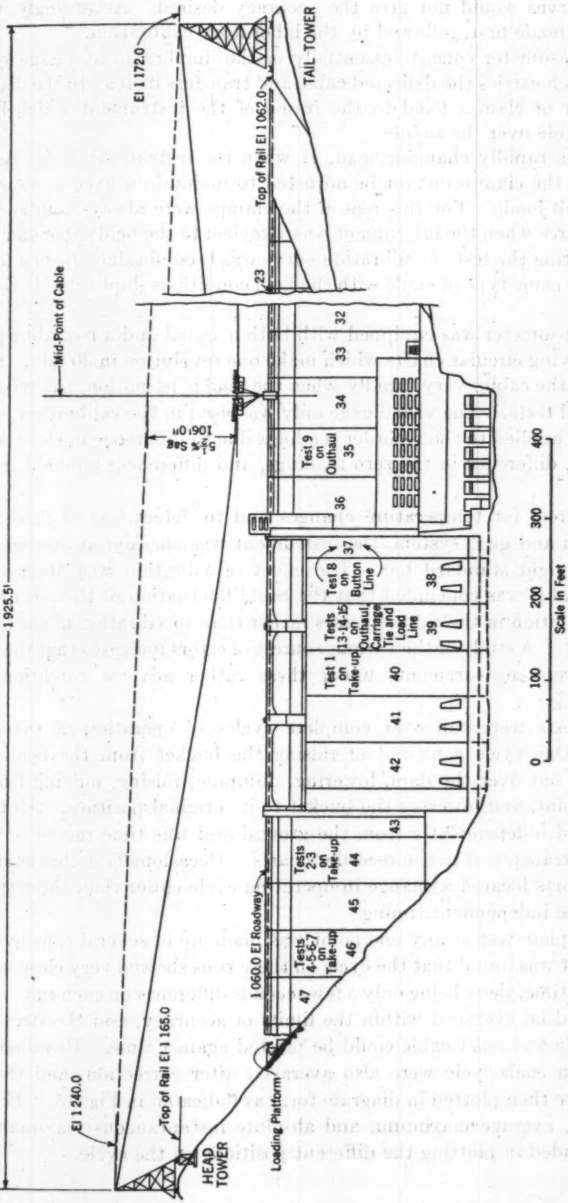

TAIL TOWER
El 1172.0
Top of Rail El 1062.0
23
Mid-Point of Cable
32
33
34
35
36
37
38
39
40
41
42
43
44
45
46
47
5½% Sag
1061 ft
Test 9 on Outhaul
Test 8 on Button Line
Tests 13-14-15 on Outhaul, Carriage Tie and Load Line
Test 1 on Take-up
Tests 2-3 on Take-up
Tests 4-5-6-7 on Take-up
1925.5'
1060.0 El Roadway
Loading Platform
El 1240.0
Top of Rail El 1165.0
HEAD TOWER
0
100
200
300
400
Scale in Feet

FIG. 14.—DOWNSTREAM ELEVATION OF NORRIS DAM, SHOWING BLOCK NUMBERS AND LOAD POSITIONS FOR ROPE TESTS (SEE FIG. 3)

bration curves would not give the accuracy desired. Accordingly, the field tests were made first, followed by the laboratory calibration.

The tensometer consists essentially of the diaphragm and gage system, a saddle which carries the deflected cable and transfers its load to the diaphragm, and a pair of clamps fixed to the frame of the instrument which bend the stressed cable over the saddle.

Under a rapidly changing load, or when the instrument is not constantly accessible, the clamps cannot be adjusted to maintain a fixed pressure on the cable for all loads. For this reason the clamps were always tightened to the proper degree when the instrument was attached to the field cable and were not altered during the test; a calibration curve was then obtained from a test specimen of the same type of cable with the field conditions duplicated in the testing machine.

The tensometer was equipped with both a visual and a recording gage, the latter carrying circular charts which make one revolution in 30 min. Since the stresses in the cables vary rapidly when the load is in motion, the recorder was used for all tests. The visual gage only was used in the calibration, so corrections were applied to the recorder readings due to difference in elevation of the two gages, difference in the zero readings, and differences inherent in the two gages.

To correct for temperature changes and to detect loss of fluid from the diaphragm and gage system, the instrument was checked at 30-min intervals against a rigid standard bar. The effect of vibration was observed rather closely, and it was concluded that the rapid fluctuation of the record pen was due to variation in the cable stresses rather than to vibration of the measuring equipment. A study of the various sources of errors indicates that the accuracy of the stress measurements under these rather adverse conditions is well within 10%.

The tests were run over complete cycles of operation of the cableway bucket. One cycle consisted of raising the bucket from the loading point, moving it out over the dam, lowering, dumping, raising, moving back to the loading point, and lowering the bucket to its original position. All the cycles were timed independently from the ground and this time record of the cycle was later transposed to the recorder charts. Occasionally a change in reading on the charts located a change in operating cycle closer than these transposed lines of the independent timing.

A complete test at any one point was made up of several runs over several cycles. It was found that the cycle on these runs showed very close uniformity regarding time, there being only a few seconds difference on each run. The time cycle could be averaged within the limits of accuracy, and the stresses in the track cable and hoist cable could be plotted against time. Readings for each position in each cycle were also averaged after correction, and the average cycles were then plotted in diagram form as indicated in Fig. 15. The average minimum, average maximum, and absolute instantaneous maximum stresses were included in plotting the different positions on the cycle.

In Fig. 11 the stresses in the track cable at the points in the cycle where they were maxima, which in each case was the position of lowering the bucket to dump, were plotted and compared with those determined theoretically for this location on the span, using the same formulas and assumptions which formed the basis for the original computations.

Tests on Take-Up Cable.—Perhaps the most consistent and interesting results and undoubtedly the most important were on the take-up tests.

Four tests were run while carrying concrete to block 46, two to block 44, and one to block 40 (Fig. 14). The tensometer was placed in 5 of the 10 take-up parts (Fig. 3) during these tests so that conditions of the tests were sufficiently divergent to establish the consistency of the results. The general cycle of these tests (see Fig. 15) established that, for several seconds after the uphaul begins, the stress in the cable increases very little over that obtained by virtue of the dead loads; and that no impact is apparent. Immediately thereafter came a sudden fluctuation of the recording needle, indicating increase in stress and the application of impact. The explanation of this action seems to lie in the fact that when load is first applied to a cableway, the first few seconds are consumed in readjusting the shape of the main cable to the new sag while the load still retains contact with the ground. Once contact with the ground is removed, impact sets in.

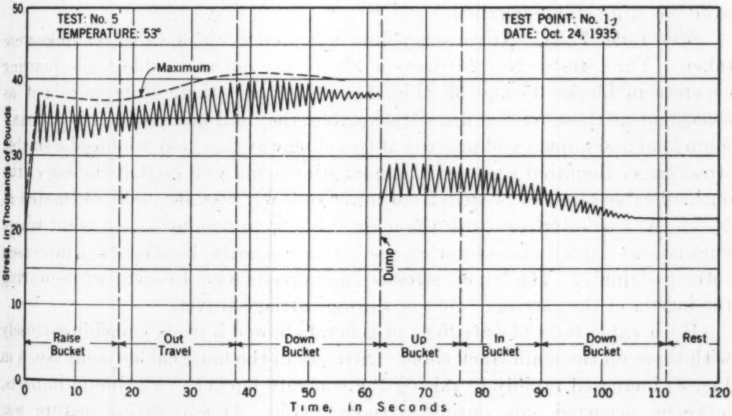


FIG. 15.—TEST OF TAKE-UP CABLE—1⅜ IN. 6 BY 19, 10-PART LINE; TEST NO. 5, TEST POINT NO. 1-*j* (SEE FIGS. 3 AND 14)

During the remainder of the uphaul, outhaul, and down travel, the stresses in the main cable increase, but with slightly diminishing impacts. The dumping point in each case is most clearly defined by a sudden stress drop with impact corresponding to a rebound. As might be expected, these stresses decrease during the return travel, and, after the return uphaul, impacts also show decreasing increments until the bucket returns to the loading point. In studying Figs. 15 and 16 it should be recalled that the load is a 6-cu yd bucket weighing 3½ tons when empty and 17 tons loaded. Since only one cable of a 10-part line was tested, the plotted stresses should be multiplied by 10 to give the stress in the track cable, neglecting friction. Fig. 16 shows a typical chart taken from the tensometer, and the cycles can be traced easily from this chart. No clearly defined variation was apparent in the stresses obtained from the different positions of the gage on different parts of the take-up line. It was found impractical to evaluate any difference in stress in these parts which might lead to a conclusion as to friction values in the sheaves.

In Fig. 11 the theoretical stresses in the main track cable were plotted for any point in the cable with the concentrated load at that point; also the theoretical stresses were plotted under the same conditions with a 25% increase in the load to allow for impact. A similar curve of these same stresses plus 25% of the stress as an impact allowance was plotted. The actual values as determined from the tests were then plotted on this same chart. It is significant that for those areas covered by the tests the computed stress plus 25% impact remained within the maximum and minimum actual stress values and closer to the former. The allowance of a 25% increase in the load for impact, as originally computed, is slightly low, and a more accurate figure for impact appears to be 25% of the computed stress.

One interesting disclosure was noted—instantaneous impacts, recorded by the needle jumping up and immediately returning, amounted to about 8% over the more stable maxima.

Other Cable Tests.—At various points indicated in Fig. 3, other readings were taken. Three tests (No. 2*a*) were made on the outhaul cable while pouring concrete in blocks 35 and 39 (Fig. 14). Although these tests were not as consistent as those for the main track cable, they did indicate points of maximum load and impact and prove that this cableway function produced a sizable stress larger than that assumed. Explanation could well be that the sag of the outhaul cable on its clear span from tower to tower was originally estimated at 5½%. As this interfered with the main cable, it was reduced to a point where fluctuations did not cause interference. Consequently, its stress was increased correspondingly. The largest stresses and impacts seem to occur when moving the inertia of the carriage—that is, during carriage travel.

Hoist cable tests (4-part line) in general showed a cycle coinciding closely with those on the main track cable. Stresses in the hoist cable (point No. 4 in Fig. 3) increased rapidly in raising the concrete bucket. Maximum impacts, however, occurred only during carriage travel. An interesting feature was that except for intensities, the same shape of curve for impacts and stresses was obtained for a 3-cu yd load and the usual 6-cu yd load.

Traversing Cable Tests.—Perhaps one of the most difficult design assumptions to confront the designer of a movable heavy-duty cableway is the pull on the anchorages in moving the towers. The tests on the traversing lines brought out some very interesting results. The upstream traversing anchorage and traversing ropes may be seen in Fig. 5.

For designing these anchorages, a pull of 80,000 lb was assumed, based mainly on experience rather than calculations. Under all ordinary conditions this seemed sufficient. The head tower, counterweighted, weighed 1,229,000 lb, and ran on 32 wheels plus 8 thrust wheels. The weight of the counterweighted tail tower was 1,253,000 lb, and it ran on the same number of wheels. No precise distribution of friction in the various wheel sets, nor of the 18-ton live load and 11-ton carriage and fall blocks, was available.

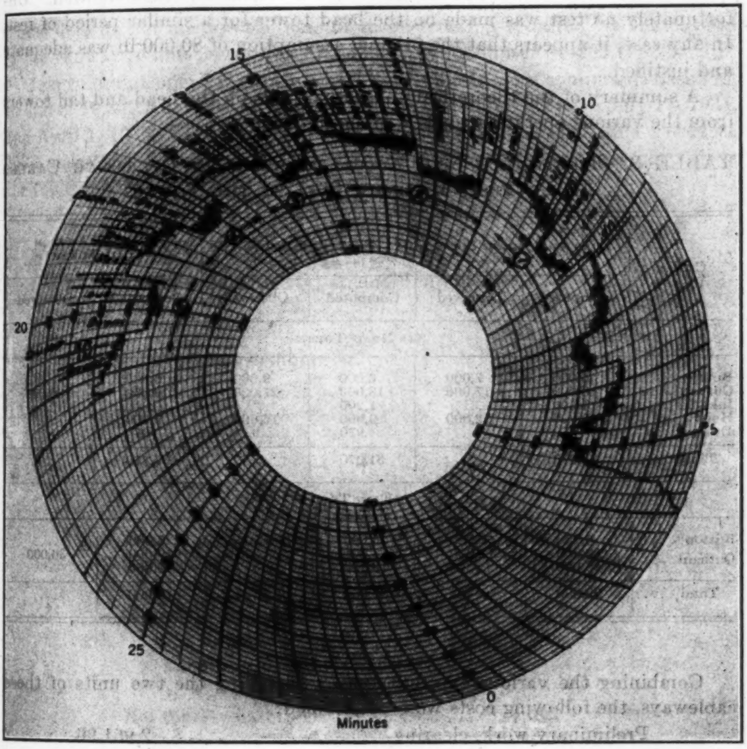


FIG. 16.—TEST OF TAKE-UP CABLE. TYPICAL CHART FROM TENSOMETER; SAME RUN AS IN FIG. 15

Tests conducted with the cableway loaded indicated 56,000-lb pull on the anchorage required to begin movement of the head tower from rest and 44,000 lb to move the tail tower under similar conditions of loading. With the cableway unloaded, movement of each tower required 44,000-lb pull on the anchorages. It is not likely that any satisfactory explanation for the discrepancy between these two tests exists, because small differences in elevation and alinement in the traversing tracks, a skewed position of the cableway, or the position of the live load may affect this movement. These tests, at points 5 and 6 as indicated in Fig. 3, were conducted during normal operations of the cableway.

Impact stresses of approximately 25% resulted, and the largest variation in stresses occurred when the initial inertia of the tower was overcome to start the tower in motion, after which the stress curve flattened out.

Another test was conducted on the tail tower after it had stood idle for two hours, and with the cableway empty. A maximum pull of 70,000 lb was required to start movement of the tower from rest under this condition. Unfortunately no test was made on the head tower for a similar period of rest. In any case, it appears that the original assumption of 80,000 lb was adequate and justified.

A summary of the theoretical and actual pulls on the head and tail towers from the various operating ropes is given in Table 8.

TABLE 8.—THEORETICAL AND ACTUAL STRESSES IN OPERATING CABLES (IN POUNDS); MAXIMUM PULL AT THE TOWERS

Cables	LOAD AT LOADING POINT		LOAD AT MIDSPAN		LOAD 140 FT FROM TAIL TOWER	
	Computed	Observed	Computed	Observed	Computed	Observed
			(*a*) HEAD TOWER			
Button......	6,000	7,000	6,000	9,000	6,000	....
Outhaul.....	13,000	17,000	13,000	27,000	19,000	36,000
Inhaul......	14,500	....	1,200	....	....	....
Hoist.......	9,900	12,000	9,900	16,000	9,900	....
Dump.......	970	....	970	....	970	....
Total	44,370	....	31,070	....	35,870	....
			(*b*) TAIL TOWER			
Button......	6,000	7,000	6,000	9,000	6,000	....
Outhaul.....	13,000	17,000	13,000	27,000	19,000	36,000
	13,000	....	13,000	....	19,000	....
Total	32,000	....	32,000	....	44,000	....

COSTS

Combining the various expenditures for installing the two units of these cableways, the following costs were determined:

Preliminary work, clearing..................	$ 2,961.96
Constructing runways..........................	34,993.40
Steel trestle..................................	19,440.26
Thrust anchors................................	16,584.67
Tracks...	13,193.27
Towers, first cost erected....................	100,318.00
Counterweights and miscellaneous tower expense	14,687.85
Hoists, track cable, carriage, operating cables, electrical equipment and controls, traversing equipment erected..................	187,613.75
Dismantling..................................	22,591.12
Sub-total......................................	$412,384.28

Salvage value (credit) from transfer of one cableway to another project and the sale of part of one cableway	96,795.79
Net cost	$315,588.49

These data do not include the cost of reconstruction required by the cave-in of the runway foundation in May, 1935, which amounted to $63,723.25.

Likewise unit operating costs on an hourly basis were computed, based on a total of 19,342.5 net operating hours for the two units, between July 17, 1934, and April 1, 1936. This number of net hours is more than in Table 4 because 508.5 delay hours are included. Subdividing into the component parts gave the following:

Operation			
Labor		$5.463	
Expense—electricity, plant, supplies, etc.		1.024	
Total operation, per hour			$ 6.487
Maintenance			
Labor			
Field		$2.508	
Shop		0.274	
Total maintenance labor		2.782	
Expense—shops, machines, etc.		0.658	
Material (original cables not included)	$2.324		
Credit—salvaged cable	0.331		
Net total—materials		$1.993	
Total maintenance, per hour			5.433
Total operating cost, per hour (depreciation not included)			$11.920

This total cost was for the period extending beyond the completion of the dam. The actual cost at the close of the job was $11.44 per hr, which does not include the cost of repairing a cave-in under the head-tower runway.

DISCUSSION

J. S. Foster,[3] **Esq.** (by letter).—Facts and computations relating to the overhead cableways used in the construction of the Norris Dam are presented concisely and interestingly in this paper, and the authors are to be congratulated for their careful and comprehensive work.

The computations for cable and rope stresses which, as the authors state, are according to accepted formulas probably have never previously been checked with the actual stresses in similar installations—certainly not in such a thorough manner. The writer believes that there is not available a tensometer registering to nearly 400,000 lb as required on this installation if applied directly to the main cable; therefore, the use of the smaller tensometer on each of the 10 "parts" of the take-up as was done by the authors should give a substantially accurate result. It is apparent, as shown by the curves in Fig. 11, that stresses at point (1) (about 32% of the span) would be somewhat less than calculated or actual results if taken at midspan, about 960 ft from the head tower.

The authors' explanation of moderate impact in lifting the load at the point of pickup is logical and is to be expected; the flexible main cable "droops," with only gradually increasing resistance to the pull of the hoist rope, until deflection of the main cable under the load at the pickup point ceases—without doubt minimizing impact as the load leaves the ground.

In comparing computed stresses plus 25% impact as shown in curve *B*, Fig. 11, with tests including impact as in curve *E*, Fig. 11, it is a question of which result (calculations or test by tensometer) is more accurate; however, it is an interesting showing, and tensometer test with the loaded carriage at rest at different points (for instance at 1, 2, 3, 4, 5, 6, and 7, or a few of them) compared with calculated stress without impact would be interesting, because it is not quite clear from Fig. 11 that such results have been obtained. The allowance made in design calculations for an impact of 25% of the live load could have been assumed only.

The cableway towers embody the Ackerman type of thrust-wheel, as described and illustrated in the paper, and their thorough and rugged construction is well shown in Fig. 6, as compared with traveling towers with inclined front track having inclined wheels and horizontal track with vertical wheels at the rear, all with single flanges. The latter type of towers was used on the cableways for Conchas Dam and other dams in the West. In calculating the horsepower required to move the traveling cableway towers with full load suspended, it is essential to consider the power necessary to accelerate not only the tower with its machinery and counterweight, but also the cable, ropes, carriage, bucket, and load from rest; but moderate speed and moderate accelerating time only are required, so that, with level track and reasonably well-lubricated track-wheel journals, required power to accelerate may be controlled. Possible misalinement of the two towers, or slight irregularity of the trackway as suggested by the authors, may account for the seemingly heavy pull required to move the towers at Norris Dam. The radial cableway towers at Conchas Dam were moved by traction, and misalinement was unlikely. These Conchas Dam towers (145 ft high, each weighing about 300 tons, including the machinery and counterweight) required only about 11 hp to move one tower at a uniform speed of about 75 ft per min; to accelerate required about 98 hp.

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[3] Lidgerwood Mfg. Co., Elizabeth, N. J.

In view of the cost of renewing the main cables, it is important that they be operated at stresses and carriage-wheel loads within proper limits; and it is obvious that they were so operated on the Norris Dam. Each main cable finished the job, handling more than 1,000,000 tons, and at least one of the original main cables was used afterward on another dam being constructed by the TVA.

The data on effect of temperature change on the length of the main cable are interesting and had not previously been published in connection with cableway work.

The tensometer tests on operating ropes shown in Table 8 do not show much variation from calculated stresses except for the outhaul rope. Variation in this rope is due to the fact that it is set up with a considerable initial tension to give necessary adhesion for driving and to avoid too much slack on the inhaul side of the rope. The conveying drum in this case is spool-shaped and the endless rope surges back and forth. This general practice is necessary on long spans to avoid the expense of very large single drums or an additional rewinding drum. Shorter spans use separate ropes with fixed ends on one drum for inhauling and outhauling. The considerable stress is also due partly to the fact that the maximum stress given occurs when the load is being hauled up the steep grade approaching the tail tower.

Apparently, the type of bucket used on Norris Dam was very successful. As a matter of fact the aerial or automatic dump bucket, using a third drum enabling the cableway leverman to open, discharge, and close the bucket from his station in the head tower, was used throughout on Hoover Dam, Parker Dam, and other important dams. This third drum, automatic dump also will be used on the cableways for building Shasta Dam.

The data on output and cost of operation are very interesting and indicate that two cableways placed concrete in one shift at the rate of 180.7 cu yd per hr which was the limit of mixer output; one cableway actually placed 162 cu yd in 1 hr, but normally worked at a continuous rate of 120 cu yd per hr.

Operating cost per ton, exclusive of depreciation, appears to be about 3½ cents, or equivalent to about 7 cents per cu yd on the basis of $11.44 per hr as given at the end of the paper.

The cost of electric current given under the heading "Operation and Maintenance: Performance"—namely 66.2 kw per hr, for each cableway—is moderate and somewhat less than on contract work on a notable installation on the west coast.

Actually the salvage value realized would probably be considerably more than $100,000 in view of the fact that the paper does not include salvage of certain material apparently unsold at the time data were compiled. On contract work, where the experienced contractor has the opportunity of continuing the use of the cableway on other contracts, no such depreciation need be considered. Obviously, the contractor who buys secondhand machinery, if it is in good condition, leaves most of the depreciation with the original purchaser; but this advantage of the secondhand buyer may easily be lost in the difference in cost of operation between out-of-date or damaged machinery and new machinery of more modern design and construction.

GORDON H. BANNERMAN,[4] M. AM. SOC. C. E. (by letter).—Congratulations are due the authors for the fine manner in which they have reported the construction of the Norris Dam cableway system. This paper has made very interesting reading, and the tests described with their results are a notable addition to the subject of modern heavy-duty cableways.

There is no question as to the desirability, and oftentimes necessity, of transmitting the horizontal thrust of the track cables to the movable tower foundations by the use of horizontal thrust wheels. This is a practical and safe method of taking care of these stresses.

Motorizing the trucks of movable towers has also greatly improved cableway designs. Another improvement has been to equip the track-cable end connections with both horizontal and vertical joints, the stresses being transmitted through eyebars and forged pin blocks. The cable tensions are taken in direct tension and shear, eliminating the undesirable U, or loop, bolt. This type of connection, provided with lubricating fittings, will dampen both the horizontal and vertical vibrations to a greater extent, and will relieve unequal stresses in loop bolt members that have given trouble.

It has been realized for some time that the actual stresses in operating ropes must be greater than the computed stresses, this difference in stresses being problematical. The stress values in Fig. 8, to the writer's knowledge, are the first actual stresses that have been determined in this type of installation. Similar tensometer tests have been made on the larger power-shovel operating ropes, and occasionally actual stresses were found to be 50% to 100% in excess of computed stresses.

At Shasta Dam, in California, the writer has collaborated with Francis T. Crowe, M. Am. Soc. C. E., and B. W. Goodenough, Assoc. M. Am. Soc. C. E., with the result that the button ropes on the 2,672-ft cableway spans have been increased to ⅞-in. diameter and the endless ropes to 1⅛-in. diameter.

The larger button ropes on these spans will more easily support the increased number of buttons necessary, and will offer more resistance to abrasion and impact as the slack rope carriers are taken off the carriage horn by the buttons. In most instances, the endless rope is set at an erection tension higher than necessary to assure the operator that no slippage will occur on the endless drum. When this high erection tension is added to the additional tensions produced by the normal operation of the cableway and the stresses due to impact, the total tension results in a low safety factor. This fact is revealed in Table 8. A counterweighted endless rope would assure safer operating conditions, but to date has not seemed practicable.

The hoist or load line receives severe impact stresses as shown in Table 8. The hook load is raised so quickly that the wires in the strands and the strands in the rope have difficulty in adjusting themselves to this impact stress. The length of hoist line from becket to load block operates at a disadvantage with no place for adjustment to be absorbed; so this portion of the rope generally fails first. The stresses here are an accumulation of torsion stresses due to stretch. Sufficient additional length is provided so that this part of the hoist line can be cut away and a new becket hitch made.

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[4] Hamden, Conn.

The life of operating rope can be increased by the use of larger sheaves. At Shasta Dam, sheaves 5 ft in diameter were adopted throughout the 2,672-ft spans. The life of track cables is definitely proportional to the individual wheel loads. The twelve wheel carriages were ample for 6-cu-yd concrete loads at Norris Dam. Carriages provided with sixteen wheels are recommended for 8-cu-yd concrete loads, particularly on the longer spans, and where the track cable at the pickup point is on a steep grade.

The pickup points for the load should be as far distant as practicable from the towers in order to keep the cable angle under the individual wheels as small as possible. At this point, the load applied may be equal to the maximum load, whereas the tension in the track cable is considerably less than maximum tension, resulting in a larger angle under the load.

The cross section of the track cable shown in Fig. 2 is of a small cable. The cable, 3 in. in diameter, used at Norris Dam, contained approximately double the number of wires shown in Fig. 2. In general, track cables deteriorate due to bending stresses and not due to abrasion or tensions. For this reason, the wires must be of such size as to resist, efficiently, the repeated bending due to the rolling load.

It is unfortunate that tests were not made on the track cable with the 17-ton load at the center of the span, as this is the only point where the tension can be determined accurately. The formulas involving Fig. 10, for deflection and tension at any other point, assume a constant length on the center line of the cable, and therefore the computed tensions will always be lower than the actual tensions.

WALTER F. WEBER,[5] ESQ., AND BLAIR BIRDSALL,[6] ASSOC. M. AM. SOC. C. E. (by letter).—By the publication of this paper, a valuable chapter has been added to the meager literature on the subject of the design, construction, and operation of cableways. In particular, the writers wish to call attention to Equation (2*a*). In general terms this equation may be defined as an approximation of the path of load curve for a single load traversing an anchored span.

The two end points and the center point of this curve are usually fixed by the conditions of the problem (see Fig. 10(*a*)). To find any other point accurately requires the tedious solution by trial and error of a pair of catenaries which intersect at the load.

The conditions of Fig. 10(*a*) establish the over-all catenary length of the cable. Since the physical characteristics of the track cable are known or assumed, the unstressed or ground length of the cable may be found. This unstressed length remains constant for all positions of the load on the span, except for changes due to variation in temperature. To obtain any point on the path of load accurately, two intersecting catenaries must be found. These catenaries have known spans, but unknown rises. By trial and error, a horizontal component of cable tension which is common to both catenaries and a sag to the intersection point must be found, such that the over-all unstressed length is correct and the algebraic sum of the vertical components of tension at the intersection point is equal to the known traveling load.

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[5] Dist. Engr. for Chicago, John A. Roebling's Sons Co., Trenton, N. J.

[6] Asst. Chf. Engr., Bridge Dept., John A. Roebling's Sons Co., Trenton, N. J.

The aforementioned accurate solution is very laborious and, if the formula published by the authors is a close approximation of the true curve, it is very valuable indeed. The writers have compared values obtained by the authors' formula with those found by the accurate method, mentioned herein, for a 150-ton cableway.

The characteristics of the cableway are as follows: $W = 400{,}000$ lb, $w = 134$ lb per ft, $s = 1{,}256$ ft, $h = 72$ ft, d (with load at center) $= 111.36$ ft, cable area $= 35.2$ sq in., cable modulus $= 17.0 \times 10^6$ lb per sq in. Using these basic data for both methods, the points given in Table 9(a) were found on the path of load curve.

TABLE 9.—ORDINATES *d* TO PATH OR LOAD CURVE, IN FEET

(*a*) LARGER VALUES OF *x*			(*b*) SMALLER VALUES OF *x*		
Values of *x*	Accurate*	Approximate†	Values of *x*	Equation (2*a*)	Writers'
0	0	0	0	0	0
55.40	27.42	27.95	1	1.15	0.92
120.00	45.64	46.37	10	7.99	7.60
184.60	58.59	60.38	20	13.56	13.32
306.47	79.62	80.51	30	18.21	18.10
392.44	91.13	91.41	40	22.32	22.31
628.00	111.36	111.36	55.4	27.95	28.04
Tan ϕ_1	0.945	1.438	* Intersecting catenaries.		† Equation (2*a*).

In this table ϕ_1 is the angle of the path of load curve at the head tower. From Equation (2*a*), tan ϕ_1 is found by placing $x = 0$ in the first derivative of d with respect to x.

Due to the large discrepancy in tan ϕ_1, some may feel a little uncertain of the values of d found by the formula, for small values of x. Its accuracy in this region may be demonstrated as follows: The writers have developed another approximate method of handling the problem, which is now considered obsolete because it is more difficult to apply than the authors' formula, and does not yield more accurate results. It is mentioned herein simply as a means of demonstrating the accuracy of Equation (2*a*). To apply the writers' method, it is necessary to find the true value of tan ϕ_1. Using tan ϕ_1 and the sag with the load at midspan, a part of an ellipse having these characteristics is then fitted to the span. Thus, the ordinates d are certain to be close to the truth for small values of x. A numerical comparison between the two methods is given in Table 9(b).

These comparisons clearly indicate that Equation (2*a*) is a very close approximation of the true path-of-load curve for an anchored span, and the writers hope that this demonstration will overcome, in this case, the reluctance with which many engineers use empirical, approximate formulas.

The writers suggest that the equation may be slightly easier to apply in the following form:

$$d = \frac{x\,h}{s} + \frac{\frac{x}{s}\left(1 - \frac{x}{s}\right)(w\,s + 2\,W)^2}{2\,t\left[w + 4\,\frac{W}{s}\sqrt{\frac{x}{s}\left(1 - \frac{x}{s}\right)}\right]} \quad \ldots\ldots\ldots\ldots (3)$$

In case it is desired to obtain a more accurate value for tan ϕ_1 than that obtained by the use of the authors' formula, for some reason such as the power requirement, or the total included angle over a tower saddle, the writers suggest the following method:

Find the characteristics of the track cable hanging under its own weight only. A close approximation of these values can be obtained by using the method described by F. C. Carstarphen,[7] M. Am. Soc. C. E.

From these characteristics find the horizontal and vertical components of the free cable tension at the support. To the vertical component add the weight of the moving load, algebraically. The quotient obtained by dividing this sum by the free cable horizontal component is a close approximation for tan ϕ_1.

In closing, the writers wish to emphasize that, where true accuracy is required, there is no substitute for the catenary solution. This requirement would occur, for instance, in the case where there is a close clearance between the moving load and some fixed object. However, in most cases, the comparisons shown in Table 9 indicate that the authors' formula is entirely satisfactory.

G. E. CATE,[8] ESQ. (by letter).—A formula for the deflection of the cable at the point of load is presented in this paper, the load being at any point other than midspan. This formula is not new, having been presented in 1909 by William Hewitt,[9] who used the principle of an ellipse with major axis equal to the span and the minor axis equal to twice the deflection at the center of the span under loaded conditions with the load at the center.

A slightly different approach to the derivation of this formula is as follows: In Equation (2a) of the paper

$$d_1 = \frac{x\,h}{s} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4a)$$

and

$$d_2 = \frac{x\,(s - x)\,(w\,s + 2\,W)^2}{2\,t\,[w\,s^2 + 4\,W\,\sqrt{x\,(s - x)}]} \ldots\ldots\ldots\ldots (4b)$$

The part d_2 of Equation (2a) applies to level spans and is approximate, but is sufficiently accurate for inclined spans if $\frac{h}{s}$ is small. The nomenclature of the paper is used in general, except that y = the deflection at any point of the load for level spans.

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[7] "Aerial Tramways," by F. C. Carstarphen, *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 875.

[8] Associate Civ. Engr., Construction Plant Division, TVA, Knoxville, Tenn.

[9] "Attributes of Curves Described by Moving Loads on Suspended Cables," by William Hewitt, *Industrial Engineering*, November, 1909.

When the load is at the center of the span (level supports), the moment for a uniform load is $\frac{w\,s^2}{8}$; and,

$$t_0 = \frac{w\,s^2}{8\,y} \quad \text{(5a)}$$

The moment for a concentrated load is $\frac{W\,s}{4}$; and

$$t_1 = \frac{W\,s}{4\,y} \quad \text{(5b)}$$

Consequently the total load is

$$t_0 + t_1 = t = \frac{w\,s^2}{8\,y} + \frac{W\,s}{4\,y} = \frac{s\,(w\,s + 2\,W)}{8\,y} \quad \text{(6)}$$

For the general case of a load at any point x, the moment for a uniform load is $\frac{w\,(s\,x - x^2)}{2}$; and

$$t_0 = \frac{w\,(s\,x - x^2)}{2\,y} \quad \text{(7a)}$$

The moment for a concentrated load is $\frac{W\,(s\,x - x^2)}{s}$; and

$$t_1 = \frac{W\,(s\,x - x^2)}{s\,y} \quad \text{(7b)}$$

Similarly to Equations (5), the total load is

$$t_0 + t_1 = t_x = \frac{w\,(s\,x - x^2)}{2\,y} + \frac{W\,(s\,x - x^2)}{s\,y} = \frac{(w\,s + 2\,W)\,(s\,x - x^2)}{2\,s\,y} \quad \text{(8)}$$

which is the general equation of the curve described by the path of the loaded point referred to rectangular coordinates with the origin at either point of support.

From Equation (8):

$$y = \frac{(w\,s + 2\,W)\,(s\,x - x^2)}{2\,s\,t_x} \quad \text{(9)}$$

Let t_0 = the horizontal component of tension due to the cable alone, which is equal to $\frac{w\,s^2}{8\,y_0}$, y_0 being the deflection at the center of the span; and let t_1 = the horizontal component of tension due to the load alone. Then, Equation (9) may be written in the following form—(in which y_0 = the deflection due to the cable alone and y_1 = the deflection due to the load):

$$y_0 = \frac{w\,(s\,x - x^2)}{2\,(t_0 + t_1)} \quad \text{(10a)}$$

and

$$y_1 = \frac{W\,(s\,x - x^2)}{S\,(t_0 + t_1)} \dots\dots\dots\dots\dots\dots (10b)$$

The total deflection equals $y_0 + y_1$; t_0 is a constant and the problem is to find t_1 in terms of known values; and t_1 varies from $\frac{W\,s}{4\,y}$ with a load at the center of the span to 0 if the load is at either support. Then if y' represents the ordinate of the intervening points of the curve,

$$t_1 = \frac{W\,(s\,x - x^2)}{y'\,s} \dots\dots\dots\dots\dots\dots (11)$$

Choosing an ellipse with a major axis equal to s and a minor axis equal to $2\,y_0$ as a gaging curve (this is not the curve of the path of the load), the equation of the ellipse is

$$\frac{x^2}{\left(\frac{s}{4}\right)^2} + \frac{y^2}{{y_0}^2} = 1 \dots\dots\dots\dots\dots\dots (12)$$

with the origin at the center of the span. Translating the origin to either support, Equation (12) becomes

$$y' = \frac{2\,y_0}{s}\sqrt{s\,x - x^2} \dots\dots\dots\dots\dots\dots (13)$$

and $\left(\text{since } t = \frac{s\,(w\,s + 2\,W)}{8\,y_0}\right)$

$$\frac{2\,y_0}{s} = \frac{w\,s + 2\,W}{4\,t} \dots\dots\dots\dots\dots\dots (14)$$

Finally,

$$y' = \frac{(w\,s + 2\,W)}{4\,t}\sqrt{s\,x - x^2} \dots\dots\dots\dots\dots\dots (15)$$

Then substituting the value of y' from Equation (15) into Equation (11), and simplifying:

$$t_1 = \frac{4\,t\,W\,(s\,x - x^2)}{s\,(w\,s + 2\,W)\,\sqrt{s\,x - x^2}} \dots\dots\dots\dots\dots\dots (16)$$

and, since (from Equation (6)) $t = \frac{w\,s\,(w\,s + 2\,W)}{8\,y}$,

$$t_0 = \frac{w\,s^2}{8\,y} = \frac{t\,w\,s}{w\,s + 2\,W} \dots\dots\dots\dots\dots\dots (17)$$

Equation (9) may be written in this form

$$y = \frac{(w\,s + 2\,W)\,(s\,x - x^2)}{2\,s\,(t_0 + t_1)} \dots\dots\dots\dots\dots\dots (18)$$

Substituting Equations (16) and (17) in Equation (18),

$$y = \frac{(w\,s + 2\,W)\,(s\,x - x^2)}{2\,s\left\{\dfrac{t\,w\,s}{w\,s + 2\,W} + \dfrac{4\,t\,W\,(s - x^2)}{s\,(w\,s + 2\,W)\,\sqrt{s\,x - x^2}}\right\}}$$

$$= \frac{(w\,s + 2\,W)\,(s\,x - x^2)}{2\,s\left\{\dfrac{t\,w\,s^2\,(w\,s + 2\,W)\,\sqrt{s\,x - x^2} + [4\,t\,W\,(s\,x - x^2)]\,(w\,s + 2\,W)}{s\,(w\,s + 2\,W)\,(w\,s + 2\,W)\,\sqrt{s\,x - x^2}}\right\}}$$

$$= \frac{(w\,s + 2\,W)\,(s\,x - x^2)\,(w\,s + 2\,W)\,(w\,s + 2\,W)\,\sqrt{s\,x - x^2}}{2\,t\,\{w\,s^2\,(w\,s + 2\,W)\,\sqrt{s\,x - x^2} + [4\,t\,W\,(s\,x - x^2)\,(w\,s + 2\,W)]\}}$$

$$= \frac{(w\,s + 2\,W)^2\,(s\,x - x^2)\,(w\,s + 2\,W)\,\sqrt{s\,x - x^2}}{2\,t\,\{w\,s^2\,(w\,s + 2\,W)\,\sqrt{s\,x - x^2} + 4\,W\,(s\,x - x^2)\,(w\,s + 2\,W)\}}$$

$$= \frac{(w\,s + 2\,W)^2\,(s\,x - x^2)}{2\,t\,(w\,s^2 + 4\,W\,\sqrt{s\,x - x^2})} \qquad \text{(19)}$$

If y is replaced by d_2 and $(s\,x - x^2)$ by $x\,(s - x)$, and Equation (4*a*) is added, Equation (19) becomes Equation (2*a*), the formula presented by the authors.

ADOLPH J. ACKERMAN,[10] M. AM. SOC. C. E. (by letter).—The authors of this paper have rendered a valuable service in describing the design, construction, and operation of modern cableways. It is seldom that information on heavy-duty construction equipment is available in such detail, and the data with respect to design, performance, and operation, including first cost and operating costs, are very valuable. The cableways used in the construction of the Norris Dam are in themselves engineering projects of sufficient importance to merit a place in the engineering literature.

The ordinary impression regarding such heavy-duty cableways is that they represent an excessive investment and as such are a questionable type of construction plant. Although this may be true for the ordinary type of construction, it is the writer's opinion that, where topography is favorable and where the volume of concrete to be handled is of substantial proportions, there is no cheaper way to build a dam than by cableway. Furthermore, it is important to recognize that cableways of small capacities and moderate cost have been used for a great variety of work in Europe, even for the construction of ships, and it is unusual that they have not found wider acceptance in the United States. Probably one of the reasons for this is that many engineers and constructors are not "cableway minded," and tend to visualize a cableway as a highly flexible and unstable unit that is not as dependable as a crane running on the firm ground. Nevertheless, those who have had experience with cableways, and have organizations of riggers and other personnel skilled in the handling of cableways, realize that the full possibilities in cableways have not been reached and that they deserve a much wider application.

The first important installation of a heavy-duty cableway, capable of handling 25 tons, was used in the construction of the Owyhee Dam in Oregon in 1929 and 1930, and in many respects this was the forerunner of a number of heavy-duty cableways built since then. Table 10 contains a list of the important cableway installations used in the construction of dams during the ten years 1930–1940.

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[10] Director of Eng., Dravo Corp., Neville Island, Pittsburgh, Pa.

TABLE 10.—HEAVY-DUTY CABLEWAY INSTALLATIONS

Dam projects	Number of cableways	Capacity, in tons	Span, in ft	Track cable diameter, in in.	Hoist power, hp	HEAD TOWER		TAIL TOWER	
						Height, in ft	Type*	Height, in ft	Type*
Owyhee	1	25	1,306	3	400	65	T	45	F
Madden	1	25	1,325	3	400	100	T	100	T
Hoover	2	25	2,575	3	500	90	T	90	T
	2	25	1,405	3	500	75	T	42	T
	1	25	1,365	3	500	98	F	16	T
Morris (Pine Canyon)	2	15	960	2¼	300	100	F	35	T
Norris	2	18	1,925	3	500	75	T	110	T
Bonneville lock and power house	2	15	1,390	3	400	145	F	75	T
Bonneville spillway	2	20	2,020	3	500	90	T	223	F
Parker	2	25	1,500	3	500	75	T	42	F
Conchas	2	15	1,650	2½	250	145	T	175	F
Hiwassee	1	18	1,575	3	500	75	T	110	T
Marshall Ford	1	25	2,100	3	500	75	T	177	F
Shasta	6	25	800 to 2,600	3	500	460	F	75 to 125	T

* T = traveling and F = fixed.

Mention should also be made of the 150-ton cableway that has been installed permanently at Hoover Dam for handling heavy equipment from the rim of the canyon down to the power-house levels on either side of the river. This is an outstanding engineering accomplishment in this field and indicates the possibilities in the use of cableways.

Concrete Buckets.—At the time the Norris Dam cableways were proposed there was considerable discussion on the question of whether the concrete should be handled in buckets of 4-yd capacity or 8-yd capacity. In several earlier cases the 8-yd buckets had developed objections because the type of bucket used discharged the concrete too suddenly and with sufficient impact to disturb the concrete below which was in the process of setting. Several installations of cableways with 4-yd buckets had proved satisfactory, but it was decided finally to compromise and use two cableways with 6-yd buckets. The use of two cableways provided an economical arrangement and increased flexibility; but, as has been demonstrated on other jobs, a single cableway with an 8-yd bucket of improved design can be used successfully in the construction of a dam as large as Norris Dam.

Since the concrete bucket on a cableway is the "business end," this part of the equipment deserves special attention. At Norris Dam efforts were made to improve the method of discharging large volumes of concrete with a minimum of segregation and impact and this led to a very useful development, originated by Ross White, M. Am. Soc. C. E.—namely, an air ram built into the bucket in such a way that, when an operator in the form applied a short air nozzle to an air socket on the bucket, the ram would do the heavy part of the work in operating the gate on the bucket. By proper adjustment of the gate the flow was controlled so that a very satisfactory discharge was developed. The bucket was so designed that it would reclose automatically while it was being hoisted, without returning to the form for latching.

Runways for Towers.—In planning the installation of a cableway, the principal problem is to find suitable sites for the tower runways. It is normally desirable to keep the span as short as possible; and under ideal conditions a flat area on each side of the canyon provides the most favorable runway sites.

However, in many cases the profile at the dam site continues to rise to a considerable height above the top of the abutment. It then becomes necessary to excavate a bench in the hillside. This means that the ground directly in front of the runway, sloping downward into the valley, introduces problems of instability with respect to supporting the horizontal forces from the tension in the track cable and operating rope. Where the bench can be excavated into solid rock, this is a less serious problem, but frequently a solid formation is not available. It may also be necessary to extend the runways by means of earth fills that have similar characteristics of instability against horizontal thrust, particularly since they are usually constructed on the downward sloping natural ground.

In the earlier types of traveling cableway towers very little consideration was given to stresses in the runways due to horizontal thrust; usually the tower was supported on ordinary railroad trucks, and the horizontal thrust was transmitted to the rails through the wheel flanges. This introduced very heavy bending moments in the wheels and frequently resulted in breakage of flanges or wheels.

An improvement over this design consisted of inclining the front wheels in the same general direction as the front part of the tower so that the wheels carried both a vertical component and a horizontal component or a resultant inclined force corresponding more or less to the inclination of the wheels. This arrangement, however, meant that the runways had to be built either of steel, on solid rock, or with heavy ties connecting the front and rear tracks to prevent spreading under the constant shifting of the wheel loads between front and rear tracks. For example, with no load on the cableway the maximum wheel load is on the rear tracks due to the counterweight. However, as heavy loads are picked up on the cableway to the point where practically all of the counterweight is lifted, the entire force—cable pull and tower weight—is balanced on the front tracks. It is interesting to note that on many European cableways the rear track is eliminated entirely and only front inclined wheels are used, the back end of the tower rocking up and down, depending on the tension in the cables.

As a further improvement in wheel arrangement and in order to make the cableway towers adaptable to a greater variety of runway conditions, the writer conceived the idea of a tower in which all horizontal thrust is taken at the rear of the tower and transmitted into the runway at that point. Since its first application at Madden Dam in the Panama Canal Zone, a number of dams have been built using this type of cableway.

The principal advantage is that the resultant inclined thrust is not transmitted into the front edge of a runway fill where it is least stable, but is directed into the rear side of the fill and the entire fill is utilized to resist the horizontal thrust. This is clearly indicated in Fig. 1, showing the location of the horizontal thrust wheels running on an earth-fill runway for the tail towers at Norris Dam. It is apparent that if the front wheels had been inclined to carry the tremendous resultant thrust without using the rear thrust wheels, the fill would have required considerable additional width. A further advantage of this arrangement of wheels is that under all conditions of cableway loading the principal runway reactions are carried at the rear. With no cableway load, when the counterweight is most active, the principal force, of course, is on the rear wheels. At full cableway load the vertical forces on the front wheels increase but none of the heavy horizontal component is transmitted to the front wheels; this remains at the rear of the runway. This means that with a dependable arrangement of tracks at the rear, the tower could still operate with considerable misalinement or settlement of the front track.

The result of all this is that the structural arrangement of the runways and of the tower is made as simple as possible. The importance of economy in the location and design of runways is apparent from the fact that in the case of the Norris Dam installation they represented $87,000 of the total cost; very little of this construction is salvageable.

The authors call attention to the advantage of introducing the horizontal thrust at the rear of a runway, and refer to a cave-in that occurred under the head tower. Referring to Fig. 1, the runway for the head tower is shown in detail. During the construction of Norris Dam it was discovered that a clay-filled cavern in the abutment area, which was being cleaned out prior to back filling with concrete, had a chimney outlet, and it so happened that one of the trestle runway bents was located directly over this chimney. In cleaning out the clay plug in the cavern below, a displacement occurred and the trestle bent suddenly sank several feet while supporting one of the head towers. Without the rear thrust-wheel anchorage this tower would have rolled down the hillside.

Radial Cableways.—In some cases the topography does not lend itself to economical construction of parallel runways for two traveling towers. In such cases, it is generally feasible to locate a radial runway for a traveling tower on one side of the river, and a fixed high tower on the other side so that the cableway serves a fanlike area which covers the entire construction site. Typical installations of this type were used in the construction of Bonneville, Marshall Ford, and Conchas dams.

At Marshall Ford Dam one 8-yd radial cableway was used, but the other projects had two radial cableways, the two traveling towers in each case traveling on the same radial track. In most cases the operating machinery was in the traveling towers, thus making the fixed tower the tail tower, which at Marshall Ford Dam was 177 ft high, and at Bonneville Dam 223 ft high. The tail towers were designed with backstays to carry the cable pull applied at the top of the towers to deadmen anchored securely in the ground some distance to the rear. In this manner the towers acted as struts and little tension was carried in any of the legs.

The anchorage of the track cable and of the backstay cable deserves special attention in such installations since the tension in the track cable is in the order of 200 to 250 tons. The problem is aggravated by the fact that with a concrete bucket traveling out and back in a 3-min to 4-min cycle, the release of load is fairly rapid (within 8 to 11 sec) and considerable bouncing of the track cable takes place due to change in load. In the aforementioned installations, this problem was handled in a very satisfactory manner by relieving the tower of most of the change in horizontal force and carrying such forces directly into the elastic backstays.

The connection at the top of the tower consisted of a pair of eyebars suspended from a horizontal pin. These bars were about 3 ft long and were free to swing in a vertical plane. The lower end of the bars carried a heavy pin, designed with a universal joint which in turn held the terminal eyebars of the main track cable. The universal joint permitted the track cable to move laterally through the arc made by the traveling tower at the other end of the span. A second pair of eyebars about 3 ft long was framed into the same lower pin and extended rearward to a connection with the backstays. In other words, the vertical eyebar hangars supported the juncture of the backstay with the track cable. Then the assembly was free to rock back and forth with changes in track-cable tension without changing the loading in the supporting tower over an excessive range, except in so far as a shifting of the resultant was represented by the swinging of the suspender bars. These eyebars were restrained to rock, only in a vertical plane, by means of rollers mounted at the outer end of the universal joint pin, and these rollers transmitted directly into the tower frame the lateral component that was introduced by the travel of the tail tower on either side of the center line.

Where the radial cableway covers a considerable arc, it may be advisable to carry two lines of backstays so that the track-cable forces are split into two components, with whatever internal angle seems most desirable.

The writer recently had occasion to explore the use of a backstay with the track cable anchored directly to the top of the tower without rocker links. Such a tower must either be strong enough as a cantilever to take the entire force or to carry part of the force within its range of deflection, the remaining force being carried by the backstay. For such a condition it was found that any kind of twisted cable or even locked coil cable has a modulus of elasticity (about 19,000,000) that is too low to permit much of the load to go into a backstay even when pre-stressed. In other words, the rigidity of the tower is ordinarily such that it tends to pick up most of the horizontal load within its normal range of deflection; and the resulting stretch in the backstays has not led to a substantial increase in stress even though the backstays have initially been pulled up until most of the sag in them has been eliminated. When it is desirable to use backstays in combination with a rigid tower, it appears advisable to make them of standard high tensile suspension bridge wires which have a modulus of elasticity of 29,000,000. This modulus is nearly the same as in the structural steel in the tower, and therefore they tend to work as a unit with the tower.

Mechanical and Hoisting Equipment.—A further interesting point with respect to arrangement of wheels on traveling towers is the drive for moving the towers. The authors have shown, in Figs. 3 and 5, a system of cables anchored at each end of the runway with a spooling type of hoist mounted in the tower which can run back and forth on these cables to move the tower. More recently traversing drives have been developed consisting of motors and gearing—that is, direct-connected to the wheels on the rear trucks, similar to the drives on modern gantry cranes. Here again the localizing of principal forces at the rear of the tower simplifies the duty of such a drive in moving the tremendous weight of a cableway tower.

The main hoist described by the authors is the usual type of hoist and motor installation. An important advance in this respect was made on three of the large cableways for Shasta Dam, where variable voltage control was applied to large cableways for the first time. Such a control utilizes a motor-generator unit in each cableway, the generator being DC and furnishing power to a DC hoist motor. R. F. Emerson, electrical engineer for the General Electric Company, Schenectady, N. Y., has shown[11] that, with this system, the bucket can be accelerated or retarded smoothly by varying the generator voltage, and higher speed of the empty bucket is attained, thus reducing the time of the return trip over that required with AC drive. It was estimated that this arrangement would result in cutting 17 sec from the cycle time for each trip of the concrete bucket, and that during the life of the job about 580 hr of working time would be saved in the operation of each cableway. The total financial advantage in this case was evaluated at about $150,000. Also, the operator can spot the bucket with greater ease; the power factor of the entire system is raised and this results in a reduction in power charges; and the life of the operating cables is increased because acceleration and retardation are much smoother. The significance of these facts is appreciated by inspecting a graphical record[12] of power consumption taken during a typical cycle of operation on the Norris Dam cableway.

Economics of Cableway Installation.—Table 3 is of considerable interest because it shows that 68,000 tons of materials other than concrete were handled by the Norris Dam cableways during the construction of the dam. The availability of hooks of such high capacity to handle the great amount of other heavy materials and equipment has an important bearing on the economics of construction plant selection, and in determining the justification of cableways over other types of cranes for the placing of concrete.

A direct comparison from actual experience with respect to a large dam built by cableways, and a very similar dam built by whirler cranes, showed some interesting results. The monthly placing rate on each dam was, at its best, about 95,000 cu yd, or an average of 3,657 cu yd per day for the entire month. The best day on each dam was almost the same—namely, 4,500 cu yd.

The costs of those items directly influenced by the plant in the production and placing of concrete were as follows:

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[11] *Engineering News-Record*, June 6, 1940, p. 116.

[12] For illustration of graphic power chart, see "Construction Planning and Plant," by A. J. Ackerman and C. H. Locher, Members, Am. Soc. C. E., McGraw-Hill Book Co., Inc., 1940, p. 64.

Item	"Cableway" dam	"Whirler" dam
Plant for aggregate, cement, and mixing	$191,700	$157,800
Concrete delivery railroad	68,500	112,000
Cableways and appurtenances	328,300	0
Whirler cranes, trestle runway, and appurtenances	0	133,000
Total	$588,500	$402,800
Number of men on all operations	42	54
Plant amortization per cu yd of concrete on 1,000,000 yd	$ 0.589	$ 0.403
Operation, labor, materials, supplies, and maintenance per cu yd of concrete	0.709	1.274
Total unit cost	$ 1.298	$ 1.677

The difference in unit costs is $0.379, or $379,000 on 1,000,000 cu yd of concrete. It should be noted that whereas the first cost of plant for the "whirler" dam was practically 19 cents per cu yd less than for the "cableway" dam, the operating costs were substantially higher and resulted in a net difference of 38 cents per cu yd in favor of the "cableway" dam. The lower first cost of plant on the "whirler" dam, using standard equipment of smaller capacity, meant using smaller buckets, more hooks, more signalmen, more transfer trains and crews, more placing crews, more maintenance on account of more machinery, runway bridge maintenance, and extra placing costs around the legs of the embedded bridge, all of which contributed to increased operating costs in the production of concrete. This demonstrates the importance of spending enough money at the beginning of a job to obtain the lowest ultimate cost.

The writer recently had occasion to analyze in greater detail the relative performance of various cableway installations as well as other types of concrete handling equipment for most of the dams built during the fifteen years since 1925, and some important conclusions were derived therefrom with respect to economics of construction plant layout as well as speed of construction for various types of dams.

Tension Waves in Cables.—The tests described in the latter part of the paper are of special significance as they offer the first dependable information regarding stresses in operating ropes and track cables for large cableways made under actual operating conditions. The great increase in stress observed in these tests as compared with stresses computed by the usual formulas indicates the importance of dynamic conditions in cable systems. The sample chart in Fig. 16 is one of a number taken at various parts of the cableway. A close inspection of the chart indicates that the stress waves in the track cable occur not merely when a heavy load of concrete is suddenly discharged, but are present at all times while a load is traveling back and forth on the cable. The stress wave has a well-defined period of 5 sec or a frequency of 12 cycles per min. It is significant that this is a stress wave running axially in the cable and not a bouncing of the cable itself. The most important conclusion from these tests was that the operating stress in the track cable is about 30% greater than that obtained by a conventional cable formula, because the cable formula is based on a static loading of the cable, whereas the increase in stress is due to dynamic loading.

In the spring of 1939, at Shasta Dam, the writer reviewed the proposed plans for a cableway layout of most unusual and ingenious form. This cableway layout consists of a fixed central tower with seven radial cableways all terminating in this tower. This scheme was devised by Francis Crowe, M. Am. Soc. C. E., and as a result of a great many studies of different types of construction plant and cableway arrangement, it was concluded that the adopted scheme would result in the concrete being placed at minimum cost and in the most practical manner. The central tower is a cantilever structure 460 ft high with a horizontal pull of 3,000,000 lb resultant at the top. This resultant is made up of a combination of loads introduced by six cableways with operating ropes, each of which develops a total pull of 400,000 to 450,000 lb. As a result of the tests made on the Norris Dam cableways, it was agreed that the applied loading on the tower for design purposes should include an allowance for dynamic loading at least 30% greater than obtains for static conditions. Considerable attention was given to the question of waves in cables and it can be readily visualized that with six or seven cableways, each carrying 8 cu yd of concrete and discharging at various intervals and sometimes simultaneously, a most complex condition of stresses is introduced at the anchorage in the top of the high tower. The spans vary in length from 2,600 ft to 720 ft and are arranged to cover the entire area of the dam within an arc of about 120°. The writer was tempted to christen this remarkable cableway arrangement "Crowe's Harp."

The phenomenon of stress waves in cables was observed more recently on a deep mine hoisting cable used in constructing one of the tunnel sections on the Delaware Aqueduct of the New York Board of Water Supply. It was found that the stress waves extended through the hoisting machinery into the 500-hp motor where they were observed by a graphical oscillograph which measured the change in magnetic field in one of the motor phase leads. The change in magnetic field was very pronounced, beyond any considerations given in the design of the motor, and might have caused serious damage if not detected in time. The period of the stress wave, as here measured electrically, coincided with an easily observed oscillation in the free inclined span cable between head frame and hoist.

R. T. Colburn,[13] M. Am. Soc. C. E., and L. A. Schmidt, Jr.,[14] Assoc. M. Am. Soc. C. E. (by letter).—This will refer to the discussion by Mr. Foster, in which he gives the power requirements for moving the cableway towers of Conchas Dam. Since one of the cableways at Norris Dam was erected, and has poured all of the concrete at Hiwassee Dam subsequent to its dismantling at Norris Dam, the writers have been able to check their measurements on the pulls against the traversing anchorages at the latter project. Fig. 17 shows the recorded pulls on the anchorage for periods when the towers stood at rest, when they were starting, and when they were moving.

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[13] Constr. Plant Engr., TVA, Knoxville, Tenn.

[14] Care TVA, Hales Bar Dam, Guild, Tenn.

No clearly defined points were determined to differentiate between the head tower and the tail tower; nor was there any consistency in the results obtained for immediate movement or after long periods of rest, and therefore the total number of readings taken for each position has been plotted and the ranges of pulls are indicated between the outside limits of the readings taken. Fig. 17 indicates that, similarly to Norris Dam, pulls of approximately 70,000 lb may be expected.

As to the horsepower requirements, several readings of the voltage and current during starting and running indicate that approximately 100 hp was required to start the head and tail towers, and that approximately 60 hp was consumed when the tower was moving.

The life of the 3-in. locked-coil track cable used at Norris and Hiwassee dams far exceeded the expectations of TVA engineers. A total of 427,090 cu yd of concrete was traversed over this cable at Norris Dam, and approximately 795,000 cu yd were hauled at Hiwassee Dam. Besides this, 33,272 tons of other materials were moved by this cableway at Norris Dam and probably twice as many tons at Hiwassee Dam.

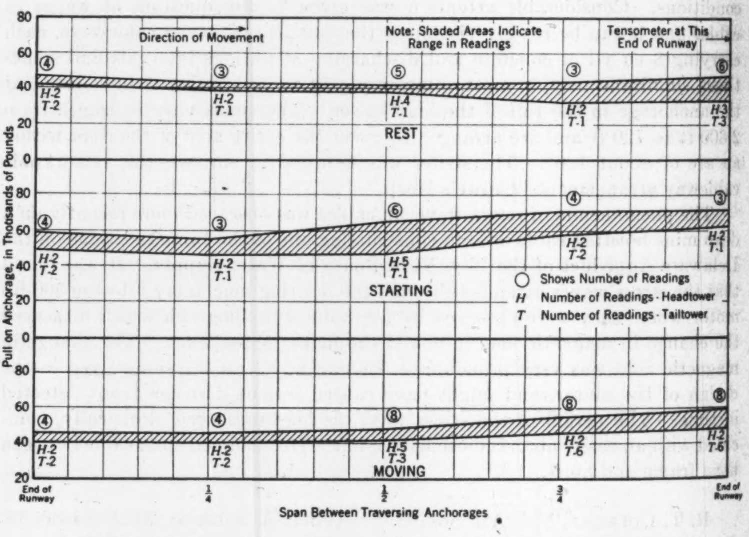

FIG. 17.—RECORDED PULLS ON TRAVERSING ANCHORAGE, HIWASSEE CABLEWAY

Maintenance, consisting of greasing and periodically turning the cable, was a large factor contributing to the longevity of this excellently built cable. The careful design of the carriage for even distribution of wheel loads to reduce bending stresses in the cable is also believed to have contributed to the lasting qualities of the cable.

The discussion by Mr. Ackerman has added greatly to the scope of the original paper by giving data on a number of older, and one of the more recent installations—namely, Shasta Dam. It is gratifying to know that advantage was taken of the results of the tests at Norris Dam in the design of this cableway installation. The discussion by Mr. Cate provided a very complete analysis and derivation of the formulas used by the writers. The additional comments in the discussion given by Messrs. Bannerman, Weber, and Birdsall were a valuable addition.

Acknowledgments.—In closing this discussion the writers wish to make the following acknowledgments: Tests were conducted by Douglas McHenry, TVA, at Norris Dam and Hiwassee Dam, who also made all the calibration tests on the tension indicator discussed under "Tests," and provided the discussion of the use of this instrument in this text. T. S. Whitehouse, construction plant electrical engineer, provided the discussion of the electrical features; and P. H. Kline, construction plant operations engineer, furnished operating data.

Special acknowledgment is made to Mr. Ackerman, formerly construction plant engineer, TVA, who supervised the design, installation, and operation of the cableway; to S. A. Parish, of the American Steel and Wire Company, and to Mr. McHenry, for suggestions offered by them after reviewing the paper. Barton M. Jones and Ross White, Members, Am. Soc. C. E., were construction engineer and construction superintendent, respectively, on Norris Dam.

The TVA design check of the cableways, the cableway layout, and runway design were made by Mr. Colburn; and the analysis and comparison of the test data by Mr. Schmidt.

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TRANSACTIONS

Paper No. 2109

MEASURING THE POTENTIAL TRAFFIC OF A PROPOSED VEHICULAR CROSSING

By N. Cherniack,[1] Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Murray D. Van Wagoner, Charles B. Winick, James H. S. Melville, and N. Cherniack.

Synopsis

The appraisal of the economic practicability of proposed self-liquidating toll crossings, financed by debenture bonds which are secured only by a lien on, and which must be serviced by, revenues from tolls, has raised technical problems of traffic and revenue estimating which are much more difficult than those encountered in the economic justification of free and tax-supported highway facilities.

This paper analyzes the potential traffic of any proposed vehicular facility into a number of elements, indicates and discusses the factors which, past experience has shown, determine the sizes of those elements, and suggests methods of estimating the elements by adequate measurements of their determinant factors. It also indicates the difficulties encountered in testing the suggested methods by checking estimated against realized traffic, after the opening of new facilities for which traffic estimates had been prepared.

Introduction

Need for Modern Vehicular Crossings.—It is common knowledge that in the two decades since the World War there has been a phenomenal growth in the use of motor vehicles for private and commercial transportation of people and freight, and that the rapidly mounting volumes of vehicular traffic have induced an expansion in highway construction. However, the much heavier expenditures required for the construction of adequate modern bridges and tunnels than for equal stretches of tributary highways has caused the building of such facilities to lag considerably behind highways. This lag, ultimately, has given rise to urgent and vociferous demands for modern river crossings.

Problem of Financing Crossings.—In the decade 1920–1930, cities and states could afford to satisfy those demands with tax-supported, toll-free bridges,

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Note.—Published in February, 1940, *Proceedings*.

[1] Statistical Analyst, The Port of New York Authority, New York, N. Y.

because they could usually borrow the necessary funds on the basis of their general credit. Consequently, in that decade of rising taxes and expanding tax yields, elaborate studies of the economic justification of highway facilities were almost superfluous. However, when tax yields began to decline, when taxpayers began to demand relief, and when governors and mayors began to find it increasingly difficult to balance state and municipal budgets, they were no longer willing (and in many cases they were unable) to borrow the necessary funds to construct even urgently needed highway bridges or vehicular tunnels which were exceedingly costly in comparison to similar stretches of highways. How to meet demands for these facilities, in the face of equally urgent state and municipal needs, created real financial problems for these officials.

Where the demand for a vehicular crossing had grown to such proportions that it became evident that to meet it would constitute a profitable venture, private companies were organized to build and operate toll crossings, usually under state franchise. A number of such toll bridges were so constructed and have been financially successful, whereas others, for various reasons, have not.

Public Financing.—At the same time, in the period under discussion, a relatively new governmental idea was being tested. It consisted of setting up a governmental agency and charging it with the duties of planning, financing, and operating such public facilities for which there was a definite and large enough demand to make the project self-sustaining in character. This agency, at first supported by its governmental parent, was to investigate the economic practicability of such facilities, and, if satisfied that they would be self-liquidating in character, was to proceed to finance such a facility on the merit of the specific project and on the sole credit of such agency.

Such a governmental agency is The Port of New York Authority. It was created by compact between the states of New York and New Jersey, which compact was ratified by Congress in April, 1921. It has planned, financed, and constructed, and now (1940) has in operation four interstate bridges connecting New York and New Jersey. It purchased from the states of New York and New Jersey, and operates, the Holland Tunnel under the Hudson River. In the midst of the economic depression, it financed the Lincoln Tunnel under the Hudson River, between West 38th Street, Manhattan, and Weehawken, N. J., at first through the Federal Works Administration, and then subsequently refinanced the loan through private investment bankers. The first tube of this tunnel was opened to traffic on December 22, 1937.

It is extremely doubtful whether the State of New Jersey, for example, could have afforded to contribute its share of the cost of constructing these four bridges and the new Lincoln Tunnel, and then to have operated them as free facilities, covering the fixed and operating charges through general taxation. More likely, these crossings would not have been built. On the other hand, this bi-state agency, by 1939, will have spent $200,000,000 for the construction of its bridges and tunnels, which have tremendously expedited the movement of vehicles, people, and freight between the two states and within the New York metropolitan area. At the same time, neither for the construction of these facilities nor for their operation, has it cost, nor will it cost, the New Jersey taxpayer (or, for that matter, the New York taxpayer), a single tax dollar.

It is the motorist who has been willing to pay for the entire cost of both construction and operation of these crossings on an "as-and-when-you-desire-to-use-them" basis. This is evidenced from the fact that the motorist patronage which these crossings have been receiving has made them, as a group, financially self-sustaining right through the most serious and unprecedented economic depression in history.

Thus it has been demonstrated definitely that, rather than be deprived of urgently needed, faster, more convenient, and safer bridges and tunnels, because they could not be built as free, tax-supported crossings, the motoring public has preferred to have such facilities built and operated as toll crossings and has given them strong financial support with their loyal patronage.

Toll Versus Free Crossings.—The appraisal of the economic practicability of self-liquidating toll crossings for financing purposes, however, has raised problems exceedingly more difficult than that of the economic and social justification of free, tax-supported highway facilities, and for the following reasons:

(1) A free facility is supported on a broad tax base; a toll facility on a narrow revenue traffic base;

(2) Taxes are mandatory, whereas tolls are optional;

(3) The success of a free facility is measured by its relief to congestion; that of a toll crossing by the traffic volume it attracts; and

(4) The economic justification of a free facility may be predicated on liberal valuations of benefits, because traffic estimates need never be checked. The economic practicability of the toll crossing must be sound because revenue estimates must meet the "cash-register test."

Elements of the Proposed Crossing's Traffic.—The staff of The Port of New York Authority, assisted by personnel of the U. S. Works Progress Administration (WPA), has made studies of vehicular toll crossings opened since 1927. Such studies have indicated that these new crossings have obtained their vehicular traffic from the following sources, from which any future proposed crossing might also derive its traffic:

(1) Existing traffic: Diversions of vehicular traffic from existing alternate routes between tributary areas; and

(2) New traffic: (*a*) "Normal" expansion of existing vehicular traffic; and (*b*) vehicular traffic "generated" by the new crossing.

MEASURING EXISTING TRAFFIC

Delimiting Its "Traffic-Shed."—To measure the existing vehicular traffic volume that could make use of the new crossing, it is necessary first to delimit "traffic-sheds" tributary to the proposed crossing, one on each side of the stream. The boundaries of passenger car and truck "traffic-sheds" may be ascertained from a time contour map constructed from data collected through a series of comprehensive automobile test runs (simulating both passenger cars and trucks) between centers of traffic density and via every alternative existing route, the travel time via proposed routes, of course, being estimated.

Measuring Its Aggregate Traffic "Reservoir."—Having delimited the vehicular "traffic-sheds," it is then necessary to determine the vehicular and passenger traffic volumes which now flow between the sheds on either side of the stream. These volumes, constituting the aggregate potential traffic "reservoir," must be determined for annual periods, since the revenues to be predicated upon these potential traffic volumes must be calculated on an annual basis.

Annual vehicular traffic volumes may usually be compiled where toll crossings are in operation. However, where part or the entire traffic "reservoir" consists of traffic over free bridges or highways, records of annual traffic volumes are usually not available. In these cases, annual volumes must be estimated from data collected for short periods. Such estimates must be prepared carefully and the probable maximum errors determined and kept within practical limits.[2]

A too optimistic estimate of the annual volume in the traffic "reservoir" may become the first serious "weak spot" in the financial forecast of the proposed crossing. Thus, a large volume of traffic in the summer should not be taken as a positive indication of a correspondingly large annual volume; nor heavy Sunday traffic to indicate heavy weekday traffic; nor peak "rush-hour" traffic a high daily volume. Seasonal, daily, and hourly "patterns" of vehicular traffic flows for the routes which form the traffic "reservoir" of the proposed crossing should be developed carefully and, with their aid, estimates of annual traffic should be prepared on the basis of sufficiently adequate samples.

It will be seen from Table 1(*a*) that an estimate based on a 1-hr count in the peak morning or evening rush hour (during which about 7% of the 24-hr traffic usually moves) would be subject to a probable maximum error of more than 50%, and that it requires a 12-hr count to reduce this error to about 15%. However (see Table 1(*b*) and 1(*c*)), if the percentage moving in the 12-hr period is determined from a 24-hr count for a specific crossing (such as the Holland Tunnel or George Washington Bridge), the error of an estimate of the 24-hr traffic of that crossing in the future, based on a 12-hr count, may be reduced to less than 5%.

Table 2 contains some daily indexes of traffic based on annual 1938 traffic. They are expressed in percentages of the five-day average, Monday to Friday, which is considered as 100%. It will be seen that: (*a*) Wednesday's and Thursday's percentages are closest to a weekday average; and (*b*) Saturday and Sunday percentages (of an average weekday) vary widely. Traffic volumes taken on normal Wednesdays or Thursdays will approximate weekday traffic volumes closely, but they will not indicate the corresponding Saturday or Sunday volumes; nor could Saturday or Sunday volumes be used to indicate weekday volumes. Consequently, a midweek day and at least a Sunday volume must always be observed to arrive at an estimate of the complete weekly traffic volume.

Table 3 shows monthly adjusted median indexes expressed in percentages of an average month (eliminating statistically the effects of growth and reflecting only seasonality) for the following types of vehicular crossing:

(*A*) 2 bridges connecting recreational with residential areas (Marine Parkway Bridge, New York City; and Buffalo Peace Bridge, Buffalo, N. Y.).

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[2] "Methods of Estimating Vehicular Traffic Volume with the Aid of Traffic Patterns," by N. Cherniack, *Proceedings*, Highway Research Board, November, 1936.

TABLE 1.—MEDIAN PERCENTAGES AND MAXIMUM PERCENTAGE DEVIATIONS FROM MEDIANS (FROM 24-HR TRAFFIC VOLUMES)

SIZE OF SAMPLE			(a) 21 SELECTED FACILITIES[a]			(b) HOLLAND TUNNEL			(c) GEORGE WASHINGTON BRIDGE		
No. of hours	Hours		Median percentage of 24-hr volume	Maximum deviation from median percentages	Maximum percentage deviation from median percentages	Median percentage of 24-hr volume	Maximum deviation from median percentages	Maximum percentage deviation from median percentages	Median percentage of 24-hr volume	Maximum deviation from median percentages	Maximum percentage deviation from median percentage
	From:	To:									
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
					WEEKDAY						
12	7 a.m.	7 p.m.	70.9	8.5	12.0	71.4	2.0	2.8	69.9	1.9	2.7
6	1 p.m.	7 p.m.	36.9	5.5	14.9	37.6	1.6	4.3	37.7	1.3	3.5
6	7 a.m.	1 p.m.	33.9	6.1	18.0	33.9	2.1	6.2	31.9	1.7	5.3
3	4 p.m.	7 p.m.	21.2	6.7	31.6	21.8	1.0	4.6	33.8	1.2	5.0
3	7 a.m.	10 a.m.	17.4	7.7	44.3	16.5	1.2	7.3	17.5	1.5	8.6
1	5 p.m.	6 p.m.	7.2	3.8	52.8	....	....	....	....	....	....
1	8 a.m.	9 a.m.	7.0	3.6	51.4	....	....	....	....	....	....
					SATURDAY						
12	7 a.m.	7 p.m.	67.4	5.2	7.7	68.6	1.6	2.3	70.5	1.3	1.8
6	1 p.m.	7 p.m.	36.0	8.7	24.2	35.8	1.2	3.4	41.4	1.1	2.7
6	7 a.m.	1 p.m.	29.5	11.1	37.6	32.4	0.7	2.2	29.4	1.0	3.4
3	4 p.m.	7 p.m.	17.5	7.2	41.1	17.2	0.7	4.1	20.8	1.5	7.2
3	7 a.m.	10 a.m.	13.5	7.0	51.9	14.4	0.5	3.5	13.9	0.7	5.0
					SUNDAY						
12	8 a.m.	8 p.m.	64.1	10.6	16.5	59.3	1.2	2.0	65.3	1.9	2.9
6	2 p.m.	8 p.m.	39.3	9.4	23.9	32.3	0.8	2.5	37.5	2.0	5.3
6	8 a.m.	2 p.m.	26.0	4.3	16.5	27.4	1.1	4.0	27.8	1.3	3.5
3	5 p.m.	8 p.m.	19.9	7.1	35.7	17.8	0.4	2.2	21.5	1.2	5.6
3	8 a.m.	11 a.m.	9.5	5.0	52.6	10.8	1.0	9.3	10.7	0.9	8.4

[a] List of crossings: Holland Tunnel, George Washington Bridge, Bayonne Bridge, Goethals Bridge, Outerbridge Crossing, Brooklyn Bridge, Manhattan Bridge, Williamsburgh Bridge, Queensborough Bridge, Victory Bridge, Pulaski Skyway, George A. Posey Tube, Sumner Tunnel, Madison Avenue Bridge, Willis Avenue Bridge, Washington Bridge, Third Avenue Bridge, McCombs Dam Bridge, University Heights Bridge, Ship Canal Bridge, 145th Street Bridge.

TABLE 2.—DAILY INDEXES OF VEHICULAR TRAFFIC FOR SELECTED PORT AUTHORITY CROSSINGS AND THREE TYPICAL HUDSON RIVER FERRIES

Day	George Washington Bridge	Holland Tunnel	Lincoln Tunnel	Bayonne Bridge	Goethals Bridge	Outerbridge Crossing	125th Street Ferry	Electric Ferries	Cortlandt Street Ferry
Monday	103.6	102.9	98.2	102.4	99.3	101.8	102.0	103.1	103.9
Tuesday	94.8	94.5	94.4	96.0	96.9	94.7	97.4	96.6	97.3
Wednesday	95.1	96.8	99.0	100.0	99.2	96.4	98.7	98.3	96.9
Thursday	94.8	97.4	99.4	99.4	99.4	98.7	99.4	99.7	99.2
Friday	111.7	108.4	109.0	102.3	105.3	108.5	102.5	102.4	102.9
Five-day average	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Saturday	137.3	113.3	102.8	102.2	107.8	140.5	119.5	92.8	70.1
Sunday (or holiday)	191.8	131.4	106.4	150.2	145.4	204.7	146.2	94.7	34.4

(*B*) 5 bridges connecting suburban residential areas (Ambassador Bridge, Detroit, Mich.; Rip Van Winkle Bridge, Catskill, N. Y.; Mt. Hope Bridge, Bristol, R. I.; Tacony-Palmyra Bridge, Palmyra, N. J.; and Outerbridge Crossing, Perth Amboy, N. J.).

(*C*) 9 bridges and 3 tunnels connecting urban residential with commercial areas (Mid-Hudson Bridge, Poughkeepsie, N. Y.; George Washington Bridge, Fort Lee, N. J.; Bayonne Bridge, Bayonne, N. J.; Goethals Bridge, Elizabeth, N. J.; Golden Gate Bridge, San Francisco, Calif.; Philadelphia-Camden Bridge, Philadelphia, Pa.; Carquinez Bridge, Vallejo, Calif.; Triborough Bridge, New York City; Henry Hudson Bridge, New York City; Sumner Tunnel, Boston, Mass.; Mersey Tube, Liverpool, England; and Antwerp Tunnel, Antwerp, Belgium).

(*D*) 2 tunnels and 1 bridge connecting commercial with industrial areas (Holland Tunnel, Jersey City, N. J.; Posey Tube, Oakland, Calif.; and San Francisco-Oakland Bay Bridge, San Francisco, Calif.).

TABLE 3.—Median Monthly Indexes[a] of Vehicular Traffic for Four Types[b] of Vehicular Toll Crossings

Month	No. of days in month	Group (*A*)	Group (*B*)	Group (*C*)	Group (*D*)
January	31	38.6	53.3	73.1	92.6
February	28	31.7	52.5	67.4	84.4
March	31	44.5	66.8	82.0	94.6
April	30	54.9	78.5	92.9	100.1
May	31	89.7	107.7	110.9	105.9
June	30	138.4	120.8	113.8	104.3
July	31	260.0	172.9	133.8	108.9
August	31	254.3	181.9	132.9	108.6
September	30	126.0	131.6	114.1	101.6
October	31	68.4	93.8	102.4	102.0
November	30	48.2	76.0	90.7	98.6
December	31	45.3	64.2	86.0	98.4
Average	30.4	100.0	100.0	100.0	100.0
Year	365	1,200.0	1,200.0	1,200.0	1,200.0

[a] Expressed as percentages of an average month considered as 100.0, reflecting varying number of days in each month and seasonality, but growth of traffic eliminated statistically.

[b] See definition of groups in the text.

Each of the four sets of indexes represents adjusted medians selected, in turn, from the median indexes (medians of monthly indexes for a series of years) of 20 individual crossings in the four groups.

It will be seen that the spring months of April and May and the fall months of September and October come closest to an average month for the year. Hence, traffic-volume observations should be made in those months, if practicable. If observations are made in the summer or winter months, care must be used to place the facility under observation, at least, into one of the four types presented herein.

Measuring Its Share of the Existing Traffic "Reservoir."—A decade or so ago, the notion was prevalent that the new toll bridges or tunnels would completely "dry up" competitive ferries, and would "inherit" all of their traffic when they ceased operating. Experience, since then, has shown that ferries continue to operate despite the competition of toll bridges and tunnels.

On the other hand, doubt has been expressed as to whether a toll bridge could divert sufficient traffic from free bridges to compete successfully with them. That doubt has been dispelled by the operation of the Triborough and Henry Hudson bridges in New York, N. Y., which have diverted substantial traffic volumes from the nearby free Queensborough and Ship Canal bridges, over the East and Harlem rivers, respectively.

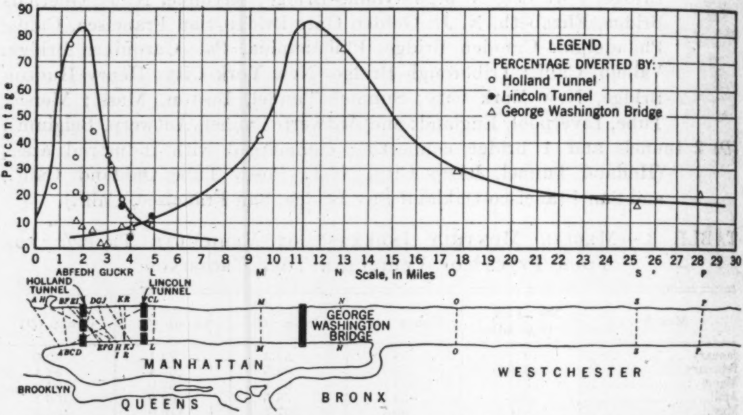


FIG. 1.—PERCENTAGES OF COMPETITIVE TRANS-HUDSON VEHICULAR TRAFFIC DIVERTED BY THE HOLLAND TUNNEL AND GEORGE WASHINGTON BRIDGE AT THE END OF THE FIRST YEAR OF THEIR OPERATION

Consequently, the problem of measuring, accurately, the share of the existing traffic "reservoir" which a proposed toll crossing can divert from competitive facilities or alternative free routes is still difficult, and requires careful study to avoid the second possible serious weak spot in the estimates.

TABLE 4.—DIVERSION EXPERIENCE FACTORS OF SELECTED TOLL CROSSINGS

Crossings	Opening date of new crossing	COMPETITIVE FERRY LINES[a]		Traffic diverted, in percentages[e]
		Number	Name	
Holland Tunnel	November 13, 1927	13	Hudson River ferries[b]	46.9
Goethals Bridge	June 29, 1928	1	Elizabeth Ferry	47.6
Outerbridge Crossing	June 29, 1928	1	Perth Amboy Ferry	65.9
George Washington Bridge	October 25, 1931	5	Hudson River ferries[c]	49.1
Bayonne Bridge	November 15, 1931	1	Bayonne Ferry	23.7
Triborough Bridge	July 11, 1936	2	East River ferries[d]	12.7
Oakland Bay Bridge	November 12, 1936	....	Ferries	61.5
Golden Gate Bridge	May 27, 1937			
Lincoln Tunnel	December 22, 1937	3	Nearby ferries	11.6

[a] Ferry routes from which bridges and tunnels diverted substantial amounts of traffic. [b] 2.5 miles or less from the Holland Tunnel. [c] 18 miles north, and 2.5 miles south, of the bridge. [d] Also Queensborough Bridge (free), 3.2 miles from the bridge. [e] Percentage of traffic volume of competitive ferries in a 12-month period preceding the opening of the new crossing, and diverted by it in its first year of operation.

Experience factors of diversions by toll crossings which have been opened are available, but such factors show a wide variation. In general, however, they indicate that the new crossing has diverted the largest percentages of

traffic: (*a*) From crossings in its proximity, and (*b*) from those crossings with the smallest favorable toll spreads (see Table 4 and Fig. 1). In Fig. 1, the letters along the base line denote approximate locations of ferry lines.

Measuring Diversions (Origin and Destination Basis).—**For the preparation of accurate estimates of the traffic divertible from alternate routes, the following information is essential:**

(*a*) Annual traffic volumes of every alternate route, segregated by "lines of travel," connecting pairs of small areas, one on either side of the river;

(*b*) Travel times and mileages between the centers of areas of traffic density;

(*c*) Tolls via all alternate crossings; and

(*d*) Physical and psychological characteristics of all alternate routes, such as type of highway, scenery, slum areas, direct or "zigzag," traffic lights, congestion, and other characteristics influencing the choice or avoidance of given routes.

Segregating the Traffic of the Crossing by "Lines of Travel."—**Traffic volumes of alternate routes are rarely available, segregated geographically. Consequently, segregations by "lines of travel" must be determined from sample origin and destination counts, compiled in sample traffic volumes between pairs of zones on either side of the river (see Fig. 2). The zones should be small enough to reveal significant differences in travel time, mileage, and other route characteristics.**

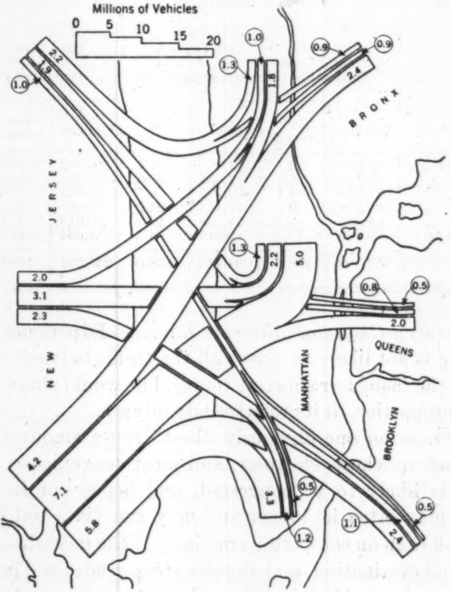
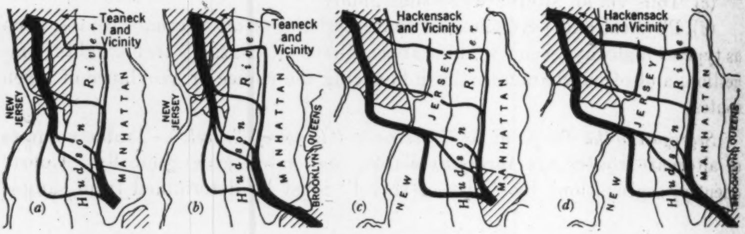


FIG. 2.—NINE TRANS-HUDSON "LINES OF TRAVEL"

Having thus segregated the annual traffic volumes of individual crossings by "lines of travel," similar "lines" between the same pairs of zones, but via alternate routes, may then be combined for all routes to determine the total travel between pairs of small areas (see Table 5). It is then necessary to determine the portions of each "line of travel," now traveling via all existing routes, that will be diverted by the proposed crossing.

TABLE 5.—ANNUAL 1935 TRANS-HUDSON VEHICULAR TRAFFIC FOR ONE OF NINE MAJOR "LINES OF TRAVEL" SEGREGATED INTO FOUR OF 56 "LINES OF TRAVEL"


Teaneck and Vicinity
NEW JERSEY
Hudson River
MANHATTAN
(a)
Teaneck and Vicinity
NEW JERSEY
Hudson River
MANHATTAN
BROOKLYN
QUEENS
(b)
Hackensack and Vicinity
JERSEY
NEW
Hudson River
MANHATTAN
(c)
Hackensack and Vicinity
JERSEY
NEW
Hudson River
MANHATTAN
BROOKLYN
QUEENS
(d)

Crossing	(*a*) Teaneck, N. J., and Lower Manhattan		(*b*) Teaneck, N. J., and Brooklyn, N. Y.		(*c*) Hackensack, N. J., and Lower Manhattan		(*d*) Hackensack, N. J., and Brooklyn, N. Y.	
	Vehicles	Percentages	Vehicles	Percentages	Vehicles	Percentages	Vehicles	Percentages
George Washington Bridge	39,657	12.7	43,683	21.4	46,192	19.4	111,555	39.7
125th Street Ferry........	39,130	12.6	42,832	21.0	23,682	10.0	31,766	11.3
42d Street Ferry..........	31,523	10.1	19,799	9.7	23,253	9.8	20,384	7.3
Electric Ferries...........	90,286	29.0	49,453	24.2	39,487	16.6	37,717	13.4
Holland Tunnel..........	61,490	19.7	29,038	14.2	70,313	29.6	52,879	18.8
Remaining 12 crossings....	49,408	15.9	19,613	9.5	34,634	14.6	26,661	7.5
Total...............	311,494	100.0	204,416	100.0	237,751	100.0	280,862	100.0

Factors Governing Choice of Alternate Routes.—Experience has shown that the new crossing is not likely to divert all the traffic between any pair of zones (even if it were the fastest or shortest route), but would share it with the alternate routes in proportion to its relative advantages.

Usually, there is no one obviously "best" route among them which is at once the shortest, quickest, cheapest, and most convenient. Frequently, the shortest route is likely to be congested, and hence not the quickest. The quickest route may often be longer and may run via a toll bridge or tunnel, with a higher toll than on competing crossings. The most direct route, between a given origin and destination, may involve steep grades and poorly paved highways and approaches. Certain other characteristics, such as directness or "zigzag," scenery or slums, parkways or congested narrow streets, and others, also influence the choice or avoidance of a route. Under such conditions motorists are compelled to weigh time, distance, tolls, convenience, directness, and other factors in selecting what each considers the "best" route at any given time.

Therefore, it is essential for the investigator to travel over all alternate routes between centers of traffic density on either side of the river, and determine travel times, mileages, tolls, and those other physical and psychological characteristics at different times. It is also desirable to consult residents of the areas for information as to unusual local conditions that might influence the choice of routes at different times of the day and in different seasons of the year.

Then again, a salesman, for example, bound for a "prospect," might place such a premium on time as to disregard a higher toll entirely; but on his return trip he might choose the cheapest, and perhaps the longest route. A truck driver might choose the shortest and cheapest route, at the expense of valuable truck time. A pleasure-bound motorist might, at times when he is "flush," disregard tolls and choose the quickest and most scenic route, whereas at other times he may choose the cheapest crossing. Thus an individual motorist may choose any route at one time and another route at another time between the same points of origin and destination. It is to be expected, therefore, that motorists in the aggregate, traveling in the course of a year between any area of origin on one side of the river and any area of destination on the other, could be found to have distributed their patronage between a number of alternative crossings. Studies have corroborated this observation and have shown that, somehow, motorists in the aggregate appear to "rate" alternative routes on the basis of their relative attractiveness along individual "lines of travel" by distributing their mass patronage as they do.

The "relative merit rating" of alternate routes for any given "line of travel" may be appraised by expressing in dollars and cents not only differences in tolls from those of a standard route, but also differences in trip time, mileage, and other physical and psychological route characteristics, and thus arriving at a relative single trip-cost factor (see Eq. 7 in the Appendix). By computing the patronages obtained by alternate routes relative to standard routes along a large number of "lines of travel" and plotting these relative patronage factors against relative "cost" factors (considering the measurable factors of tolls, running time, waiting time, distance, and their monetary evaluations), it was found that the two factors (relative "cost" and relative patronage) were related. This relationship could best be expressed by Eqs. 6 and 7 in the Appendix. The "scatter" of the data points on such charts indicated, however, that other factors (not directly or as readily measurable as are the factors of distance, running time, waiting time, and tolls) also affected the choice or avoidance of any individual route, under given circumstances, and hence its "relative merit rating." Such other factors may be safety, travel habit, avoidance of industrial or slum areas, attractiveness of scenic parkways, etc. Their effects were determined, as a group, after allowance had been made first for the effects of the measurable factors. They may be referred to as "preference" or "prejudice" factors and are represented in Eq. 7 as another cost factor (ΔC_p) which diminishes or augments the combined toll, travel time, and distance cost differences.

In Eq. 6, whereas the cost factor (ΔC) reflects the quality of the crossing, the discount factor (d) reflects the keenness of competition from its competitors. The greater the number of competitor crossings, the closer together they are, and the easier it is (by reason of the street and highway layouts) to switch from an original intention of using one crossing to that of using an alternate without undue "backtracking," the keener is the competition and hence the greater the discount factor (d).

It is probably true that very few motorists actually compute, in dollars and cents, the value of each trip between traffic centers and via different crossings. However, they probably do have a sense of value which governs their choice of possible routes between their origins and destinations on different occasions, and on the basis of this sense of value motorists in the aggregate apparently evaluate running and waiting time and distances in some proportion to each other and to the tolls, the evaluations of which may be determined in any particular study by means of origin and destination data, auto test runs, and data on tolls and other route characteristics.

By reason of these relationships (Eqs. 6 and 7), it is possible to establish an approximate measure of the "relative merit rating" of any proposed crossing which would also reflect its probable relative patronage provided its "cost" factor (ΔC) (in Eqs. 6 and 7) and the discount factor (d in Eqs. 6) could be determined from observed data.

Estimating Divertible Traffic Shared by the New Crossing.—The "relative merit ratings" of existing crossings may be determined in two ways: (*a*) From the manner in which any given "line of travel" is now distributed among them, and (*b*) from their route characteristics. The "relative merit rating" of the proposed crossing, on the other hand, must be determined by comparing its probable route characteristics with those of existing routes. By determining the "relative merit rating" of the proposed crossing (either by judgment or by the use of Eqs. 6 and 7 in the Appendix), it is possible to estimate its probable share of any given "line of travel" by computing the ratio of its "relative merit rating" to the sum of the ratings of all its competitors (see Eq. 5 in the Appendix).

For example, Col. 3, Table 6, shows the annual volume of trans-Hudson vehicular traffic moving along the "line of travel" between Passaic, N. J., Clifton, N. J., and vicinity, on the west side of the Hudson River, and the Manhattan area, 72d Street to 125th Street, east of the Hudson River, distributed among several individual crossings and the remaining crossings combined. The percentage shares handled by these crossings are shown in Col. 4 (also see Eq. 3 in the Appendix). Considering the crossing handling the largest share of this "line of travel" as having a rating of 100, the merit ratings of the individual alternate crossings are obtained by computing the ratios of their patronage to that of the best crossing. Col. 6 shows differences in travel cost, obtained by evaluating running-time differences shown in Col. 7 at 1 cent per min, waiting-time differences shown in Col. 8 at 2 cents per min, and distance differences shown in Col. 9 at 3 cents per mile, and then adding, algebraically, the toll differences shown in Col. 10 (also see Eq. 7 in the Appendix). In Col. 11 the ratings of existing crossings are shown transcribed from Col. 5. The rating of the proposed crossing shown as Item 9 in Col. 11 was obtained on the basis of judgment, using Eqs. 6*a* and 7 in the Appendix as a guide, and by comparing the cost differences of the proposed crossing with those of the existing crossings shown in Col. 6. In Col. 12 is shown the probable percentage share which each crossing (both existing and proposed) would have obtained had the new crossing been in operation in 1935. These shares were computed by dividing the rating of each crossing by the sum of the ratings of all crossings (existing and proposed); see Eq. 5 in the Appendix. Col. 13 shows the probable redistribution of the existing traffic on the basis of the percentage shares shown in Col. 12. Thus, it will be seen that along this "line of travel" the proposed crossing would have obtained 15,481 vehicles, had it been opened in 1935.

TABLE 6.—DISTRIBUTION OF ANNUAL 1935 TRANS-HUDSON PASSENGER-CAR TRAFFIC ALONG ONE "LINE OF TRAVEL," AND THE ESTIMATED REDISTRIBUTION HAD THE LINCOLN TUNNEL ALSO BEEN IN OPERATION IN 1935

			UNDER EXISTING CONDITIONS			DIFFERENCES FROM "POPULAR" CROSSING					ASSUMING LINCOLN TUNNEL IN OPERATION[f]		
Item No.	Crossing (1)	Symbol (2)	Annual number of passenger cars (1935) (3)	Percentage of total (4)	Rating factor[d] (percentages) (5)	Net cost, in cents[e] (6)	Running time, in minutes (7)	Waiting time, in minutes (8)	Distance, in miles (9)	Toll, in cents (10)	Rating factor (11)	Percentage of total (12)	Number of vehicles (13)
1	125th Street Ferry[a]	M	23,312	32.5	100[d]	....	41	7	11.8	25	100	25.5	18,276
2	George Washington Bridge	GWB	18,447	25.7	79	+16	− 5	−7	+3.3	+25	79	20.1	14,405
3	Holland Tunnel	HT	11,374	15.9	49	+48	+13	−7	+8.0	+25	49	12.5	8,959
4	West 42d Street[b]	L	9,031	12.6	39	+30	+15	+1	+1.6	+ 8	39	9.9	7,095
5	Electric Ferries	R	4,436	6.2	19	+30	+17	0	+2.8	+ 5	19	4.8	3,440
6	West 23d Street[c]	K	2,115	3.0	9	+37	+14	+5	+3.3	+ 3	9	2.3	1,648
7	Remaining crossings (7)	....	2,954	4.1	13	....	....	....	....	....	13	3.3	2,365
8	Existing crossings	....	71,669	100.0	308	....	....	....	....	....	308	78.4	56,188
9	Lincoln Tunnel	....	....	....	....	+10	−10	−7	+3.2	+25	85	21.6	15,481
	Total of Items 8 and 9	....	....	....	....	....	....	....	....	....	393	100.0	71,669

[a] For 125th Street Ferry—for this "line of travel," the "most popular" crossing—the actual time-distance and tolls are shown. [b] To West Shore Railroad, New York Central System. [c] To Delaware, Lackawanna and Western Railroad. [d] The rating factor of the crossing that handled the largest volume of traffic along an individual "line of travel" is considered as 100%. [e] Net cost, computed at 1 cent per min of travel time, 2 cents per min of waiting time, and 3 cents per mile of distance. [f] Redistributed annual 1935 passenger-car traffic.

By summarizing the portions of all "lines of travel" (into which the entire trans-Hudson traffic volume had been divided and which the proposed crossing would divert from all existing alternate routes), the total annual volume of divertible traffic in the base year may thus be obtained.

Implications of This Method of Estimating Diversions.—From the fact that the share of traffic of any "line of travel" which a crossing obtains is determined from the ratio of its rating to the sum of the ratings of all crossings (see Eq. 5 in the Appendix), it will be seen that any crossing's share thus depends upon the following factors: (*a*) The quality of the crossing itself, as reflected in its merit rating; (*b*) the number of its competitors, as reflected by the number of ratings by which its rating is divided to obtain its share; and (*c*) the respective qualities of its competitors, as reflected in the numerical values of their ratings.

Thus, the number and value of the merit ratings of individual competitive crossings determine the manner in which traffic along any individual "line of travel" is distributed among all crossings competing for each "line." For the same reason, traffic along different "lines of travel" is usually distributed differently among all crossings.

Changes in the factors that affect the ratings of any one or more of a group of competitive crossings will bring about a redistribution, from time to time, among the competitive crossings of the traffic along the "lines of travel" affected. Since crossing ratings are affected by changes in net costs via individual routes (see Eq. 7 in the Appendix), changes in ratings may be brought about by: (1) Revision in tolls (ΔT in Eq. 7); (2) changes in travel times ($\Delta T_r; \Delta T_w$ in Eq. 7) or distances (ΔD in Eq. 7) between origins and destinations, as a result of highway improvements; (3) changes in convenience, scenic attractiveness, slums, industrial areas, and other travel characteristics (ΔC_p); and (4) changes in economic conditions (ΔC_r; ΔC_w; ΔC_d; ΔT in Eq. 7).

Thus, for example, changes in tolls on any one or more of a group of competitive crossings will change the differences in costs between those crossings and the standard crossing, and not the cost differences between the remaining crossings and the standard, thereby raising the ratings of the former and, consequently, increasing the shares of traffic of the former on all "lines of travel." Similarly, the construction of a new highway would reduce travel times via only one or more of a group of competitive crossings, thus reducing net cost differences from the standard on those crossings and not on others. The effect would be to increase the ratings of the one group of competitive crossings and not those of the others, and to increase, especially, the shares of those "lines of travel" that were affected by the new highway.

In times of business depression, furthermore, motorists' evaluation of differences in time, distance, and convenience is reduced considerably, whereas differences in tolls loom large, thus bringing about higher ratings, and hence greater patronage on the lower toll, but slower, less convenient crossings.

To summarize, then, the share of traffic along any given "line of travel," which any existing crossing obtains, or any new one would obtain, is determined at any given time by: (*a*) Its own quality, based upon the motorists' evaluation of its relative travel characteristics; (*b*) the aggregate number of its competing crossings; and (*c*) their qualities. The share of traffic is determined at different times by: (*d*) Changes in its own or its competitors' relative travel characteristics, such as toll, travel time, distance, and convenience differentials, from those of some standard crossing; (*e*) changes in motorists' evaluation of those travel characteristics; and (*f*) a widening of the choice of routes.

In actual practice, the difficulties of forecasting the share which a new crossing would divert from existing crossings lies in the changes in the different factors that are constantly taking place, the lack of data (which usually are costly to obtain) required to establish the changing travel characteristics of competitive routes, and the changing monetary values placed by motorists upon travel characteristics.

The major difficulties in checking estimates of divertible traffic from each "line of travel" are due to the fact that, after the opening of the new

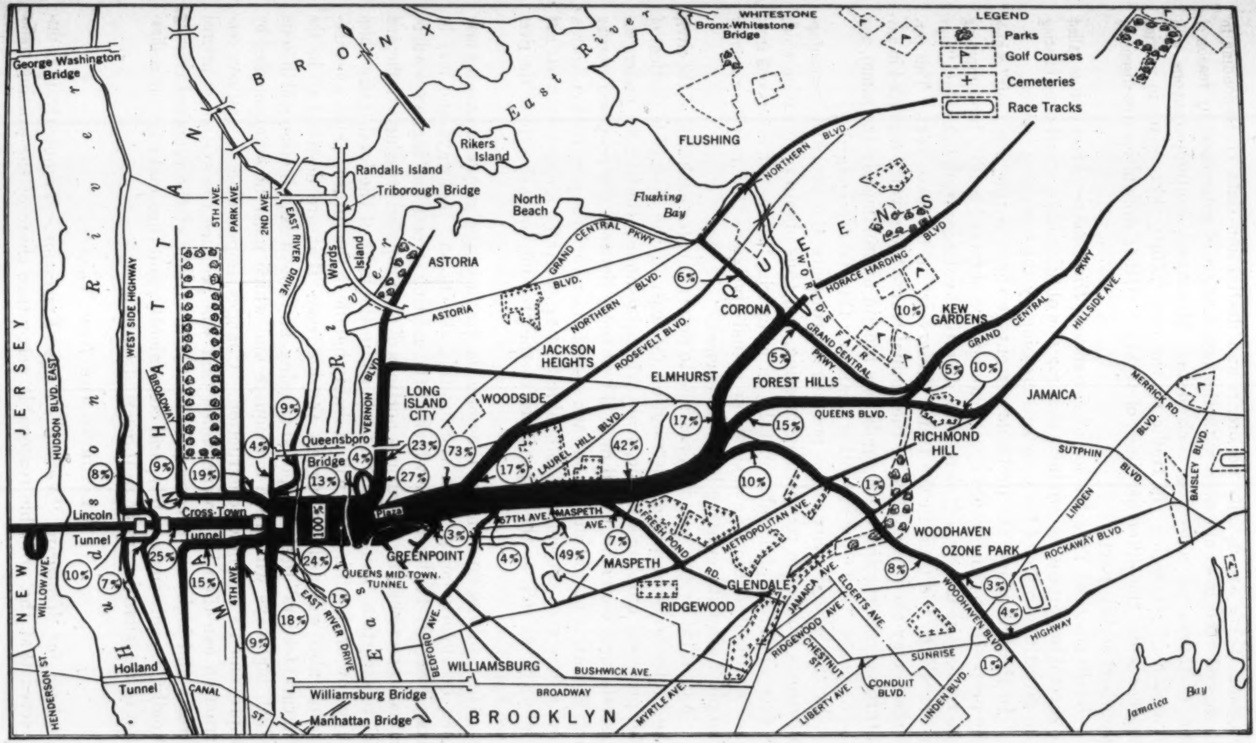

LEGEND
Parks
Golf Courses
Cemeteries
Race Tracks
WHITESTONE
Bronx-Whitestone Bridge
B R O N X
East River
George Washington Bridge
Randalls Island
Triborough Bridge
Rikers Island
North Beach
Flushing Bay
FLUSHING
Wards Island
ASTORIA
ASTORIA BLVD.
GRAND CENTRAL PKWY
NORTHERN BLVD.
ROOSEVELT BLVD.
JACKSON HEIGHTS
CORONA
Q U E E N S
WORLD'S FAIR
HORACE HARDING BLVD
KEW GARDENS
GRAND CENTRAL PKWY.
ELMHURST
FOREST HILLS
JAMAICA
HILLSIDE AVE.
MERRICK RD.
SUTPHIN BLVD.
LINDEN BLVD.
BAISLEY BLVD.
QUEENS BLVD.
RICHMOND HILL
WOODSIDE
LONG ISLAND CITY
LAUREL HILL BLVD.
VERNON BLVD
Queensboro Bridge
Plaza
100%
GREENPOINT
QUEENS MID-TOWN TUNNEL
57TH AVE.
MASPETH AVE.
MASPETH
FRESH POND RD.
METROPOLITAN AVE.
RIDGEWOOD
GLENDALE
JAMAICA AVE.
ELDERTS AVE.
RIDGEWOOD AVE.
CHESTNUT ST.
WOODHAVEN
OZONE PARK
ROCKAWAY BLVD.
WOODHAVEN BLVD
HIGHWAY
SUNRISE
CONDUIT BLVD.
LIBERTY AVE.
LINDEN BLVD.
Jamaica Bay
MYRTLE AVE.
BUSHWICK AVE.
BROADWAY
BEDFORD AVE.
WILLIAMSBURG
B R O O K L Y N
Williamsburg Bridge
Manhattan Bridge
EAST RIVER DRIVE
M A N H A T T A N
5TH AVE.
PARK AVE.
2ND AVE.
4TH AVE.
BROADWAY
WEST SIDE HIGHWAY
Cross-Town Tunnel
Lincoln Tunnel
Holland Tunnel
CANAL ST.
Hudson River
HUDSON BLVD. EAST
WILLOW AVE.
HENDERSON ST.
NEW JERSEY
6%
10%
5%
10%
5%
17%
15%
42%
10%
1%
8%
3%
4%
1%
73%
23%
17%
27%
4%
3%
4%
49%
7%
9%
4%
13%
9%
19%
8%
25%
15%
24%
1%
18%
9%
10%
7%

FIG. 3.—ESTIMATED DISTRIBUTION OF QUEENS MIDTOWN TUNNEL TRAFFIC ALONG EXISTING AND PROPOSED ARTERIAL ROUTES

crossing, there are usually no origin and destination data corresponding to those collected in connection with the preparation of estimates, by reason of the costliness of collecting such data. Moreover, contemporaneous with diversions, "generation" of traffic by the new facility takes place in varying degrees along the different "lines of travel." Other difficulties are discussed elsewhere in this paper.

By-Products of This Method of Estimating Diversions.—Attention is called to several important by-products that may be obtained as a result of estimating the divertible traffic of the proposed crossing by the foregoing method.

In the first place, the probable percentage share of the "reservoir" which the proposed crossing would divert can now be obtained readily by computing the ratio of its integrated, elemental, divertible volumes from individual "lines of travel" to the total in the traffic "reservoir" in the base year. This percentage may then be applied to the annual traffic "reservoir" compiled in any future year (assuming basic conditions remain approximately the same), and the divertible traffic for any other year may thus be determined.

In the second place, it is possible to compute the probable percentage diversions from each competitive crossing, and thus obtain information which is exceedingly helpful, subsequently, in comparing estimated diversions from each competitive crossing against the actual diversions that may be currently compiled after the crossing has been opened.

As a third by-product, it is possible to determine, for each side of the river, the probable geographical distribution (at least of the divertible portion) of the proposed crossing's traffic, from which, assuming that traffic prefers the fastest, most convenient, and most direct route between the proposed crossing and the origin or destination on each side of the river, it is possible to prepare a traffic flow map showing the probable routing of traffic along the streets and highways, as well as the approaches to the crossing on each side of the river (see Fig. 3).

The resulting flow lines of the potential traffic of the proposed crossing may then be superimposed upon the existing streets and highways and may be examined: (*a*) To determine which of the existing streets and highways will be called upon to handle the various portions of the proposed crossing's traffic; and (*b*) to ascertain their adequacy to do so, in the light of the local traffic which they now are handling and will probably handle in the future. Such an examination also will usually suggest the highways that are destined to be the "natural feeders" to the proposed crossing, and a field examination will indicate which highways will prove inadequate and which must be supplemented by improvements and new construction. Changes, improvements, and new construction can then be recommended that will produce the system of arterial streets and highways necessary to handle the traffic of the proposed crossing rapidly, expeditiously, safely, and conveniently, and thus make for the realization of its potential traffic as forecasted (see Fig. 4).

MEASURING THE NEW TRAFFIC

Need for Measuring "Normal" Traffic Expansion.—Frequently it is discovered that the probable reasonable percentage share of the existing traffic

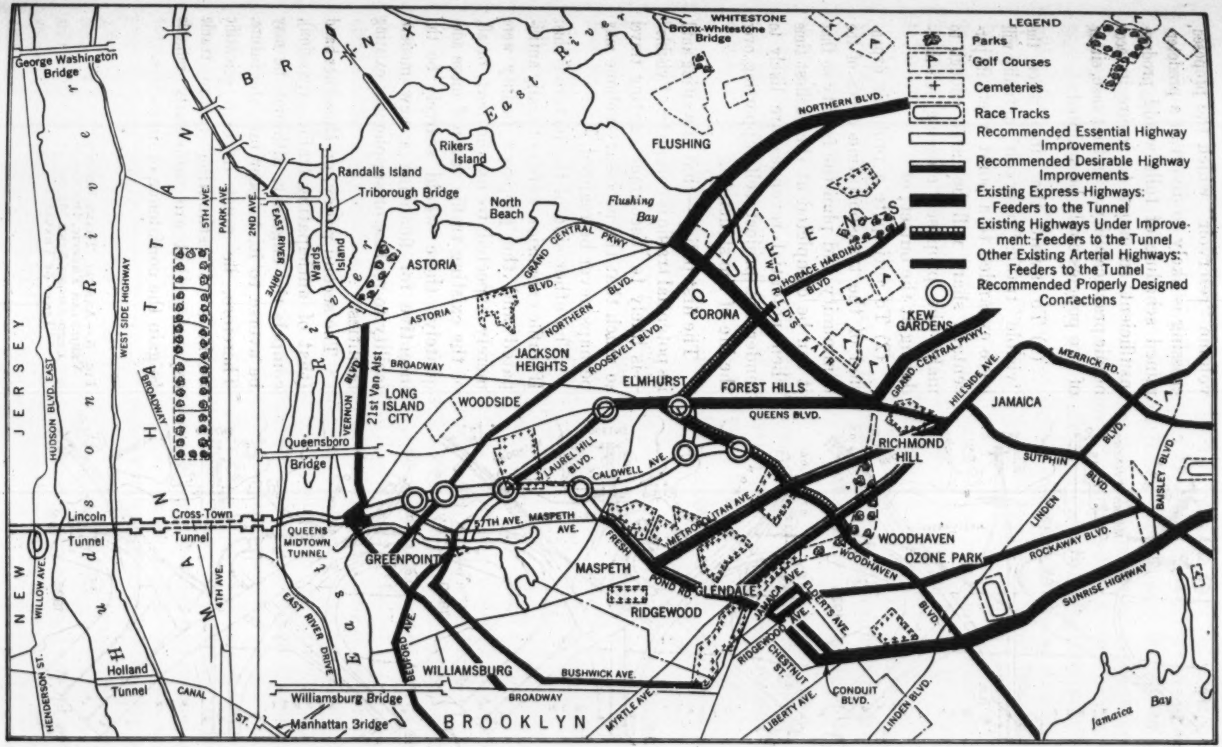

LEGEND
Parks
Golf Courses
Cemeteries
Race Tracks
Recommended Essential Highway Improvements
Recommended Desirable Highway Improvements
Existing Express Highways: Feeders to the Tunnel
Existing Highways Under Improvement: Feeders to the Tunnel
Other Existing Arterial Highways: Feeders to the Tunnel
Recommended Properly Designed Connections
NEW JERSEY
Hudson River
MANHATTAN
BRONX
QUEENS
BROOKLYN
East River
George Washington Bridge
Lincoln Tunnel
Cross-Town Tunnel
Holland Tunnel
QUEENS MIDTOWN TUNNEL
Queensboro Bridge
Williamsburg Bridge
Manhattan Bridge
Triborough Bridge
Bronx-Whitestone Bridge
WHITESTONE
Randalls Island
Wards Island
Rikers Island
North Beach
Flushing Bay
Jamaica Bay
HUDSON BLVD. EAST
WILLOW AVE.
HENDERSON ST.
WEST SIDE HIGHWAY
BROADWAY
5TH AVE.
PARK AVE.
2ND AVE.
4TH AVE.
CANAL ST.
EAST RIVER DRIVE
ASTORIA
LONG ISLAND CITY
WOODSIDE
JACKSON HEIGHTS
ELMHURST
CORONA
FLUSHING
FOREST HILLS
KEW GARDENS
RICHMOND HILL
JAMAICA
WOODHAVEN
OZONE PARK
GLENDALE
RIDGEWOOD
MASPETH
GREENPOINT
WILLIAMSBURG
VERNON BLVD.
21st Van Alst
ASTORIA BLVD.
GRAND CENTRAL PKWY
NORTHERN BLVD.
ROOSEVELT BLVD.
QUEENS BLVD.
LAUREL HILL BLVD.
CALDWELL AVE.
57TH AVE.
MASPETH AVE.
FRESH POND RD.
METROPOLITAN AVE.
WOODHAVEN BLVD.
JAMAICA AVE.
ELDERTS AVE.
RIDGEWOOD AVE.
CHESTNUT ST.
CONDUIT BLVD.
LIBERTY AVE.
LINDEN BLVD.
MYRTLE AVE.
BUSHWICK AVE.
BEDFORD AVE.
HORACE HARDING BLVD
WORLD'S FAIR
GRAND CENTRAL PKWY.
HILLSIDE AVE.
MERRICK RD.
SUTPHIN BLVD.
LINDEN BLVD.
ROCKAWAY BLVD.
SUNRISE HIGHWAY
BAISLEY BLVD.

FIG. 4.—EXISTING AND RECOMMENDED ARTERIAL HIGHWAYS IN THE BOROUGH OF QUEENS SERVING THE QUEENS MIDTOWN TUNNEL

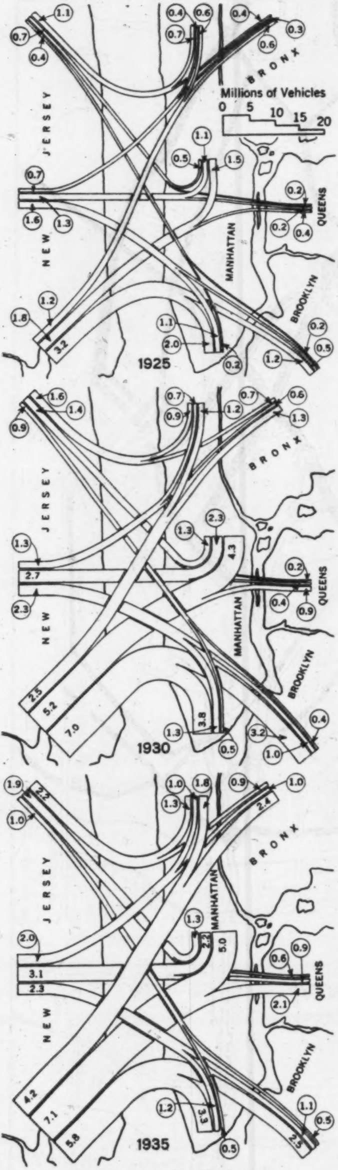

Millions of Vehicles
0 5 10 15 20
BRONX
NEW JERSEY
MANHATTAN
QUEENS
BROOKLYN
1925
1.1
0.7
0.4
0.4
0.6
0.7
0.4
0.3
0.6
1.1
0.5
1.5
0.7
1.6
1.3
0.2
0.2
0.4
1.2
1.8
3.2
1.1
2.0
0.2
1.2
0.2
0.5
1930
1.6
1.4
0.9
0.7
0.9
1.2
0.7
0.6
1.3
2.3
1.3
4.3
1.3
2.7
2.3
0.2
0.4
0.9
2.5
5.2
7.0
1.3
3.8
0.5
3.2
1.0
0.4
1935
1.9
2.2
1.0
1.0
1.8
1.3
0.9
1.0
2.4
1.3
2.2
5.0
2.0
3.1
2.3
0.6
0.9
2.1
4.2
7.1
5.8
1.2
3.3
0.5
1.1
0.5
2.5

FIG. 5.—ANNUAL TRANS-HUDSON VEHICULAR TRAFFIC, DISTRIBUTED AMONG NINE MAJOR "LINES OF TRAVEL"

volume "reservoir" which the proposed crossing is likely to divert at a predetermined schedule of tolls would produce insufficient revenues to insure its economic practicability. In that case, either of two policies could be adopted:

(1) To await the time when the "visible" traffic "reservoir" actually will have expanded to a point where its percentage share will be large enough to make the proposed crossing self-liquidating at the outset; or

(2) To anticipate a probable expansion of the transportation needs of the community and plan the facility so that it can be completed at the earliest time when potential revenues are likely to render the project self-supporting over the life of the structure.

The first policy—that of waiting until its potential traffic is completely "visible" —is likely to discourage existing travel so much that the necessary volume may perhaps never be realized.

The other side of the picture is this: If traffic expansion were not really anticipated and the proposed facility were merely expected to handle part, or all, of the existing traffic (even if more adequately) there would scarcely be the incentive for financing a new modern facility to replace or supplement existing facilities.

The second policy, on the other hand (that of anticipating traffic expansion), compels the engineer, although he may be averse to it, to peer into the future. Therefore, he must develop scientific methods of estimating future traffic expansion that are sufficiently reliable to gain the confidence of financiers.

Empirical Projection of Traffic.—Any method of estimating future trends must rely to a greater or lesser extent on past experience, modified to embody the probable effects of known or estimated future conditions. It would also be desirable, of course, if the different traffic volumes originating in the several areas tributary to the proposed crossing could be obtained for a series of years. Such data, however, are usually not available. Fortunately, in connection with studies of the George Washington Bridge and Lincoln Tunnel, and with the aid of WPA personnel in recent years, origin and destination data have been collected and compiled in 1925, 1930, and 1935. These data permit one to obtain a glimpse of the radically different rates of expansion which occur in traffic volumes originating in different areas, some of which are tributary to the new facility, and some of which are not (see Fig. 5).

Where such data are not available, the nearest approach to these data which have been compiled in recent years is the total annual traffic volumes of alternate crossings, tapping areas of which only some are common to those tributary to the proposed crossing.

Prior to 1930, when economic depressions appeared to have had no effect on traffic expansion, annual rates of expansion could be derived from such annual traffic volume series, and were expressed in several ways, such as:

(*a*) Constant average annual traffic volume increments per year (straight line) (see Eq. 8 in the Appendix);

(*b*) Equivalent average percentage of the preceding year's volume (straight line on logarithmic paper or compound interest curve) (see Eq. 9 in the Appendix);

(*c*) Variable annual volume increments dependent on available margins of capacity ("Pearl Reed" curve) (see Eq. 10 in the Appendix); and

(*d*) Other more complicated mathematical equations.

Past rates, expressed empirically, could be used either unadjusted or adjusted, and applied to past volumes to project them into the future (see Fig. 6).

Yardsticks of Traffic Expansion.—The foregoing empirical projections related traffic growth to the mere passage of time and thus impelled the inevitable assumption of continual growth. They were thus manifestly defective. It was obviously more desirable to relate traffic growth to what might be termed "traffic determinants," which would permit anticipation of shrinkages as well as expansions in traffic volumes. Several traffic determinants readily suggest themselves: Population, motor vehicle registration, opening of new crossings, economic conditions, and other expansion and shrinkage factors.

Population.—Population tributary to the crossing is obviously a fundamental determinant of traffic. Nevertheless, as a factor in vehicular traffic expansion in the past, it has perhaps been least important, other factors having eclipsed its effects (see Fig. 7). Moreover, since population census data are available only once in every 10 years, estimates available during the intercensal years are too unreliable to be of value as quantitative aids in projecting traffic into the future. Even under a stationary population, vehicular traffic may still expand substantially under the effects of greater purchasing power available for adult recreational activities, of which motor travel is one of the most universal.

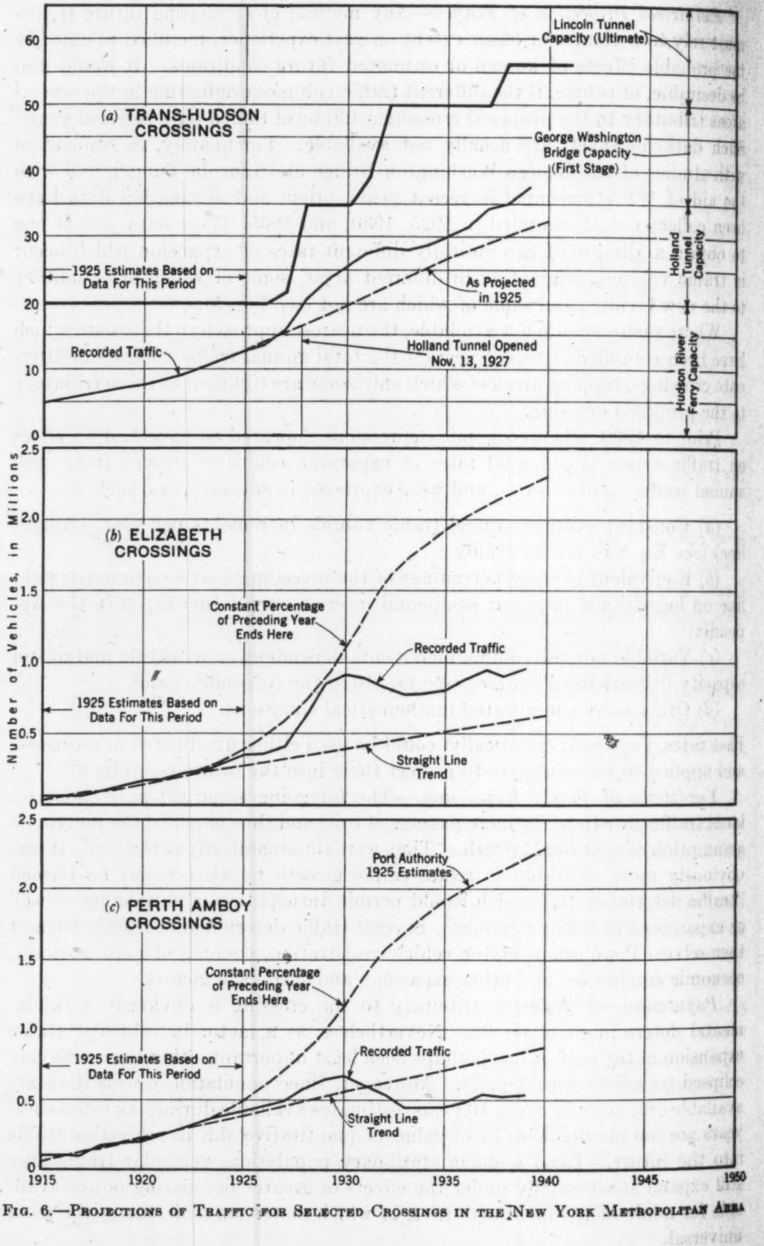


FIG. 6.—PROJECTIONS OF TRAFFIC FOR SELECTED CROSSINGS IN THE NEW YORK METROPOLITAN AREA

Motor Vehicle Ownership.—A much more intimate relation exists between growth of motor vehicle registrations in the areas tributary to the proposed crossing, and that of vehicular traffic changes. After all, a new car owner does mean a new motorist on some road or crossing, at some time. Besides, growth of motor vehicle registration also reflects growth of population completely. It must be emphasized, however, that before motor vehicle registrations may be used to forecast vehicular travel, future registrations themselves must be forecasted. Sometimes it is less difficult to forecast traffic itself than its determinant—vehicle registrations.

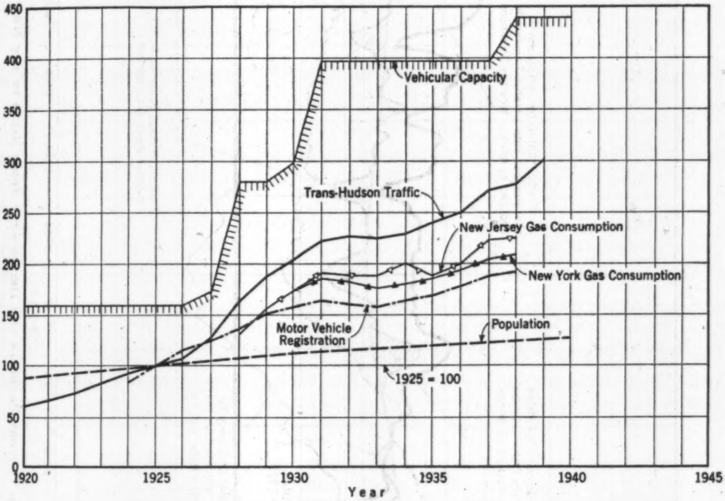


FIG. 7.—ANNUAL TRENDS OF TRANS-HUDSON VEHICULAR TRAFFIC AND RELATED FACTORS

Analyses of past trends in traffic and registrations indicated that traffic had expanded approximately in proportion to motor vehicle ownership. A comparison of trends of traffic and gasoline consumption, however, indicated that traffic expansion followed even more closely the expansion in gasoline consumption, because it reflected not only car ownership, but car usage as well (see Fig. 7).

Furthermore, in periods following the opening of new crossings, there occurred noticeable sudden "step-ups" in traffic volumes greater than those warranted by either the expansion in motor vehicle ownership or in gasoline consumption. These extraordinary volume jumps, largely attributable to increased vehicle usage or trip frequencies, were analyzed to determine their basic causes.

Economic Conditions.—Prior to 1930, the substantial upward trends prevailing in the two preceding decades largely offset any effect that economic conditions might have had on vehicular traffic expansion. Since 1930, rates of expansion have been stabilized and thus changes in traffic have reflected,

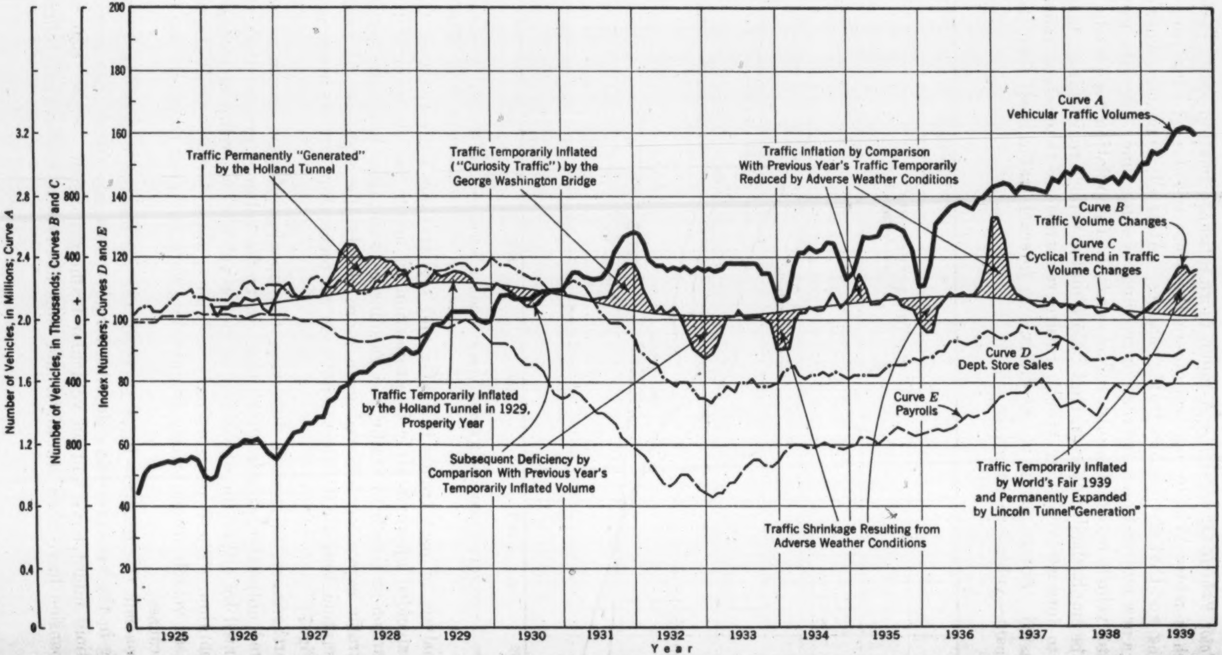


FIG. 8.—TRAFFIC OVER HUDSON RIVER CROSSINGS COMPARED WITH FACTORY PAYROLLS AND DEPARTMENT STORE SALES IN THE NEW YORK METROPOLITAN DISTRICT

more and more, the pronounced effects of economic conditions. Studies have shown that the rate of expansion of trans-Hudson vehicular traffic volumes, for example, was related to contemporary economic conditions, as reflected by factory payrolls and department store sales of the New York metropolitan district as will be seen from the curves in Fig. 8 which are defined as follows:

Curve (*A*)—vehicular traffic volumes—three month moving averages of seasonally adjusted monthly trans-Hudson vehicular traffic volumes.

Curve (*B*)—traffic volume changes—numerical changes in vehicular traffic volumes shown in Curve (*A*), from the same month in the previous year.

Curve (*C*)—trend in traffic volume changes—estimated cyclical trend obtained by spotting the crests and troughs in Curve (*B*) and fitting arcs of sine curves between them.

Curve (*D*)—department stores sales—three month moving averages of seasonally adjusted indexes (1923–1925 = 100) of department store sales in the New York Metropolitan District.[3]

Curve (*E*)—payrolls—three month moving averages of seasonally adjusted indexes (1925–1927 = 100) of New York City factory payrolls.[4]

At the time vehicular traffic volumes are projected into the future, it is nearly impossible to foresee the probable major changes in local economic conditions. These relationships, however, have served to explain the rate of traffic expansion in the past, and suggest that greater allowances in traffic trends will have to be made in the future for the effects of business conditions.

Extraordinary Traffic Expansion from Increased Vehicle Usage.—In the past, traffic has expanded approximately in proportion to motor vehicle ownership, except in periods following the opening of new crossings, when there were decided "step-ups" in traffic greater than warranted by the expansion in motor vehicle ownership.

The "step-ups" in travel attributable to increased vehicle usage (because of the added convenience of travel which followed the opening of the new crossing) could be measured by allowing for the expansion in motor vehicle ownership. In the case of traffic on the highways, vehicle usage could be measured by the average annual gasoline consumption per registered car in the entire state. In the case of vehicular crossings, a much more specific measure of usage was obtained in terms of travel frequencies, expressed as the annual number of cross-river trips per registered car, computed by dividing the annual traffic of groups of crossings serving tributary areas by the annual registrations in such tributary areas (that is, areas that furnished about 85% of the crossing's traffic). In both cases, allowance was thus made for changes in motor vehicle registrations, resulting in a measure of the effects of other factors on vehicle usage.

In the determination of the increased usage of a local facility, gasoline consumption data have not proved to be adequate for two reasons: The figures usually cover an entire state and, consequently, do not reflect local conditions; and these data have become available only in recent years.

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[3] *Monthly Review of Credit and Business Conditions*—Federal Reserve Bank, New York City.

[4] *Monthly Survey of Current Business*—U. S. Dept. of Commerce.

Factors Affecting Increased Travel Frequencies.—Studies revealed that several factors were responsible for the "step-ups" that followed the opening of new crossings: (*a*) Removal of capacity limitations; (*b*) conversion of pedestrian, railroad, and trolley passengers to bus traffic; and (*c*) increased convenience of travel.

Removal of Capacity Limitations.—In the first place, where capacities of highway facilities were largely absorbed, such limitations were retarding normal expansion. Consequently, when these limitations were removed by the opening of new facilities, traffic volumes suddenly expanded to levels which they probably would have reached had there been no such limitations to retard "normal" expansion.

Increased travel frequencies, caused by the removal of capacity limitations of existing vehicular facilities, could be estimated from general analyses of past trends in travel frequencies under varying vehicular capacities, after measures of the annual working capacities of crossings had been established carefully (see Table 7).

TABLE 7.—MARGINS OF CAPACITY VERSUS TRAVEL FREQUENCIES VIA HUDSON RIVER CROSSINGS

Year (1)	Margin of capacity (percentages) (2)	Trips per vehicle registered[a] (3)	Year (1)	Margin of capacity (percentages) (2)	Trips per vehicle registered[a] (3)	Year (1)	Margin of capacity (percentages) (2)	Trips per vehicre registered[a] (3)	Year (1)	Margin of capacity (percentages) (2)	Trips per vehicle registered[a] (3)
1915	75.2	....	1921	58.4	20.2	1927[b]	25.2	14.9	1933	43.5	20.8
1916	72.4	....	1922	54.1	17.9	1928	40.8	17.5	1934	42.3	20.7
1917	69.3	30.2	1923	47.6	16.8	1929	31.8	18.4	1935	39.4	21.1
1918	67.0	28.3	1924	41.3	15.9	1930	26.4	19.3	1936	36.9	20.8
1919	64.1	24.9	1925	37.3	14.7	1931[c]	25.5	20.1	1937[d]	31.5	21.4
1920	61.9	22.0	1926	31.6	14.0	1932	42.9	20.7	1938	38.1	21.3

[a] Annual number of one-way trans-Hudson trips per vehicle registered in tributary counties east and west of the Hudson River. [b] Holland Tunnel opened November 13, 1927. [c] George Washington Bridge opened October 25, 1931. [d] Lincoln Tunnel opened December 22, 1937.

Bus Passengers.—In the second place, new bus lines were established, routed via the newer, faster crossings, and transporting former railroad, trolley, and ferry passengers (see Table 8). Increased travel frequencies, created by increased bus travel resulting from the shifts of railroad, trolley, and ferry passengers to interstate buses, could be determined by comparing rail and rapid transit fares with bus fares and travel times, allowing, in addition, for the extra convenience of door-to-door bus service as against station-to-station rail service.

Increased Convenience of Travel.—Simultaneously, additional "step-ups" in passenger car and truck traffic occurred, created by the extra travel conveniences afforded by these modern facilities. New feeder routes and faster crossings reduced travel time by from 15 min to more than an hour. These reductions in travel time, in effect, brought residential areas on one side of the river much closer to the working, shopping, and theatrical districts on the other

TABLE 8.—GROWTH OF TRANS-HUDSON BUS TRAFFIC, IN THOUSANDS

Year	Trans-Hudson ferries	Holland Tunnel[a]	George Washington Bridge[b]	Lincoln Tunnel[c]	All Hudson River crossings
1925	71.1	....	....	....	71.1
1926	154.7	....	....	....	154.7
1927	270.6	6.5	....	....	277.1
1928	238.1	232.1	....	....	470.2
1929	319.4	356.9	....	....	676.3
1930	314.7	462.4	....	....	777.1
1931	303.3	465.0	27.7	....	796.0
1932	163.6	390.0	238.4	....	792.0
1933	136.1	379.9	368.1	....	884.1
1934	145.0	344.1	439.3	....	928.4
1935	143.1	367.9	507.1	....	1,018.1
1936	129.8	378.3	592.7	....	1,100.8
1937	150.9	331.6	643.3	3.7	1,129.4
1938	61.4	307.3	633.5	186.1	1,188.3
1939	....	323.4	661.4	473.7	....

[a] Holland Tunnel opened November 13, 1927.
[b] George Washington Bridge opened October 25, 1931.
[c] Lincoln Tunnel opened December 22, 1937.

TABLE 9.—VEHICULAR TRAFFIC "GENERATION" EXPERIENCE FACTORS FOR SELECTED TOLL CROSSINGS

New crossings	Opening date	Competing crossings traffic[b]	"GENERATED" TRAFFIC EXPRESSED IN TERMS OF:					Average toll on new crossing, in cents
			Estimated annual volume of generated traffic	Annual toll cost, in thousands of dollars	PERCENTAGES OF:			
					Traffic diversion[b]	Competitive crossing traffic[f]	Crossing's traffic in first year[g]	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
(a) CROSSINGS IN THE PORT OF NEW YORK								
Holland Tunnel	November 13, 1927	11,686,300	4,250,000	2,189	125	39.3	55.5	51.5
Goethals Bridge	June 29, 1928	409,973	365,422	190	187	89.1	64.5	52.0
Outerbridge Crossing	June 29, 1928	480,751	184,632	96	58	38.4	36.5	52.0
George Washington Bridge	October 25, 1931	4,987,000	2,225,000	1,168	65	45.0	39.5	52.5
Bayonne Bridge	November 15, 1931	583,160	336,420	173	244	57.7	70.9	51.5
Triborough Bridge	July 11, 1936	12,214,000	5,115,000	1,329	105	42.0	51.0	26.0
Lincoln Tunnel	December 22, 1937	5,245,000[c]	1,190,000	655	195	22.7	66.1	55.0
(b) CROSSINGS ELSEWHERE IN THE UNITED STATES								
Philadelphia-Camden Bridge	June 30, 1926	5,600,000	4,446,000	1,300	131	80.0	56.7	28.1
Carquinez Bridge	May 21, 1927	625,000	389,000	350	53	62.0	34.9	90.0
Tacony-Palmyra Bridge	August 15, 1929	376,228	875,900[d]	315	234	232.8[d]	70.0	36.0
Oakland Bridge Golden Gate Bridge	November 12, 1936 May 27, 1937	6,087,000	8,798,000	4,399	239	144.5	70.5	50.0
(c) ONE CROSSING IN EUROPE								
Queensway Vehicular Tunnel[a]	July, 1934	1,150,334	1,971,763	647[e]	145	171.4	59.1	55.5

[a] Between Liverpool and Birkenhead, in England. [b] Competing crossings traffic in the year preceding the opening of the new crossing. [c] Three nearby ferries only. [d] Includes an amount diverted from the lower Delaware River ferries and Camden Bridge that was difficult to determine. [e] This is the equivalent of 129,445£ sterling, at the exchange rate of about $5.00 per £. [f] Percentage of traffic of competitive crossings for the year preceding the opening of the new crossing. [g] Percentage of the new crossing's traffic in the first year of operation.

side. This increased speed, convenience, and safety of the new routes stimulated passenger car travel. Increased truck movements followed in the wake of the greatly increased passenger car travel in connection with such consumer industries as bakeries, laundries, etc.

Traffic "Generation" Experiences of Toll Crossings.—To determine the factors affecting increased passenger car and truck travel frequencies, several approaches were used. The first approach was to bring together a summary of "generation" experiences for a number of toll crossings in the United States and one in Europe (see Table 9).

The traffic "generation" experiences of these crossings, in the first year of their operations, may be summarized as follows:

(*a*) The volume of traffic "generated" varied from 185,000 to 8,800,000 vehicles;

(*b*) Additional transportation cost, paid for in tolls for this extra travel, varied from about $96,000 to almost $4,400,000;

(*c*) For every one hundred vehicles diverted from existing facilities, from 53 to 244 additional vehicles were "generated";

(*d*) For every one hundred vehicles handled by competing crossings in the year preceding the opening of the new one, the new crossing "generated" from 38 to 233 additional vehicles; and

(*e*) For every one hundred vehicles handled by the new crossing in the first year of its operation, from 35 to 71 vehicles constituted traffic "generated" by the crossing.

In all cases, except one, the new crossing was a toll bridge or tunnel, and the old competitive crossings were invariably toll ferries. In one case (the experience of the Triborough Bridge) the crossing developed its "generated" traffic in competition with free bridges. It is of especial interest to note that this experience (that of a toll bridge in competition with free bridges) is strikingly similar to the experiences of the other toll crossings in competition with toll ferries.

As in the case of any other empirical experience factors, however, the query in connection with these cases naturally arises as to what extent they would be applicable to future crossings elsewhere. Consequently, further analyses were made of some of the causative factors responsible for "generation" of traffic by new crossings to allow for specific factors that might be peculiar to a future crossing.

Quantitative Measures of Causative Factors of Expansion in Travel Frequencies.—An examination of travel frequencies between nearby and distant areas revealed the fact that the closer the areas, the greater the intensity of travel. New crossings, with their new approach highways, in effect, brought traffic centers closer together, in point of time, and thereby increased travel frequencies.

A further comparison of travel frequencies via toll-versus-free crossings (after allowing for distance and time) revealed the fact that travel frequencies via toll crossings were lower than those via free crossings. Consequently, it became necessary to determine a single measure incorporating trip time, mileage, and other route characteristics, as well as tolls. This was accomplished by expressing, in dollars and cents, not only tolls, but also trip time, mileage, and other route characteristics (the last, although difficult to evaluate, must be allowed for), and thus arriving at a single trip-cost factor. Trip costs thus determined accounted for variations in travel frequencies and yielded this general rule: The lower the total trip cost, the greater the travel frequency (see Fig. 9).

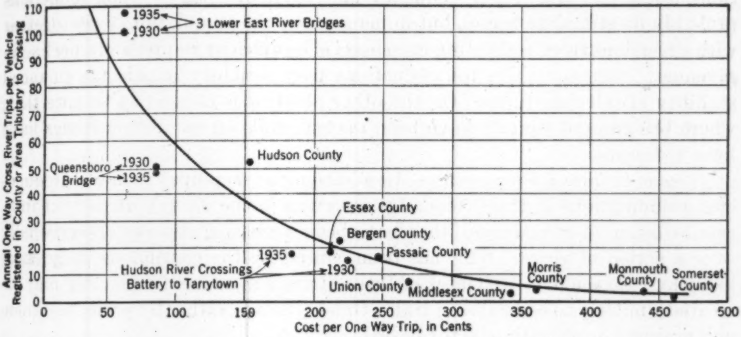

FIG. 9.—RELATION BETWEEN ANNUAL TRIP FREQUENCY AND THE COST OF A ONE-WAY CROSS-RIVER TRIP

It thus became evident that—(*a*) by reducing distance, running time, waiting time, or inordinate delays due to congestion, (*b*) by increasing the convenience of travel through the construction of additional crossings or new highways, or (*c*) by improving and integrating existing routes so as to eliminate confusing turns and "zigzags," thus making them more direct—travel frequencies could be expanded.

It should be emphasized, however, that the first bridge or tunnel which supplements a ferry service usually stimulates travel frequency substantially because, by eliminating the usual waiting time between ferry sailings, the bridge or tunnel thereby reduces travel time considerably. Subsequent competitive bridges or tunnels, on the other hand, usually will not stimulate travel frequency to the same extent as the previous crossings, even if increasing delays begin to be encountered at older fixed crossings, because these delays are usually not as great as those at intermittent crossings.

Conversely, by saddling traffic with additional costs—not only out-of-pocket costs, such as parking fees, mileage taxes, etc., but also subjective costs like economic losses imposed by customs barriers or other stringent governmental regulations, as well as undue delays on streets and highways, or, in general, increased difficulties and inconveniences of travel—travel frequencies can be expected to shrink.

The method of estimating changes in travel frequencies, therefore, resolves itself into determining trip costs properly before and after the opening of the new crossing, and consists of: (*a*) The inclusion of all the pertinent route characteristics; and (*b*) a proper evaluation of these characteristics on a dollar-and-cents basis (see Eq. 11 in the Appendix).

The present state of research indicates that a net change of one cent in trip cost (considering costs of all route characteristics) would produce a 1% change in travel frequency in the opposite direction. It is obvious that to sense the variations in the subjective values that motorists place on route characteristics at different times is an exceedingly difficult task, requires much constant research, and at present is at best an approximation.

Toll Reductions.—From the foregoing, it may be suggested that toll reductions would increase travel frequency and stimulate travel. Toll reductions probably do stimulate travel, but in many cases the stimulation, even together with diversions from competing crossings, is insufficient to prevent a decline in revenues. Consequently, toll reductions may seriously impair the financial stability of toll structures. On the other hand, it is of interest to note that, where bridges and tunnels have been opened, tolls on competing ferries have been reduced.

Trends in Travel Frequencies.—In addition to "step-ups" in travel frequencies, examinations of their trends for crossings in the New York district indicated that, in most instances, they have been increasing steadily and gradually over a period of years. This has been caused by the continuous program of highway expansion. Estimates of future trends in travel frequencies may be treated similarly to estimates of traffic trends (that is, either by projecting them on empirical or causative factor bases).

CONDITIONS FOR REALIZING POTENTIAL TRAFFIC

Highways Must Support Crossing.—In weighing the relative merits of several alternate routes, motorists invariably consider them in their entirety from origin to destination. Therefore, the advantages of routes consist of two parts: (*a*) Those inherent in the crossing itself; and (*b*) those which are a part of the streets and highways along the routes between origin and destination, as well as the approaches and immediate approach connections to the proposed crossing. Consequently, as far as the motorist is concerned, a river crossing is only part of his complete route.

If the proposed crossing is to realize the potential traffic indicated by the technical methods described, therefore, the following conditions must be in existence when the crossing is opened to traffic:

(*a*) An adequate system of streets and arterial highways tapping its tributary areas and leading the traffic conveniently and directly to the crossing;

(*b*) Not only an adequate system of street and highway approaches in the vicinity of its plazas to handle its peak traffic, but one that is superior to those in the vicinity of the terminals of its competitor crossings; and

(*c*) Properly designed plazas on both sides of the river to keep traffic moving continuously via the crossing in peak periods.

Moreover, if the proposed crossing is to attract traffic from such of its tributary areas in which its own inherent advantages are nullified by street and highway conditions, it is obvious that such disadvantages must be removed by street and highway improvements. In addition, however, improvements must be so designed that they will not improve all other alternative routes equally, but will give the routes via the proposed toll crossing decided time, distance, and convenience advantages over the then existing alternate routes and competitive crossings.

Plaza Designs.—In considering designs of plazas and approach connections, the following are a few of the conditions that may be encountered and which are of sufficient importance to require careful consideration.

Lane capacities of "signal-controlled" streets are reduced more or less drastically from what they would otherwise be under continuously moving traffic, whereas the crossing is designed for continuous traffic flow. The plazas serve as temporary storage areas, and convert intermittent traffic flow on to the plaza into continuous feeding of the crossing. In addition, the street lanes must also handle local traffic as well as crossing traffic. Each crossing lane, therefore, will usually require several street lanes, controlled by traffic lights, to which to deliver, or from which to be fed by, its traffic. Consequently, a careful examination must be made of the existing street layouts and of their effective capacities to handle their share of local traffic and at the same time, adequately, to diffuse and feed the traffic of the proposed crossing without limiting its capacity.

Thus, to provide sufficient capacity to feed the Lincoln Tunnel its traffic, and then diffuse it, it was necessary to build two new streets on Manhattan Island, one running north and south between Ninth and Tenth avenues, the other parallel between Tenth and Eleventh avenues, and between 34th and 42d streets. A similar plan is to be followed on the east side of Manhattan in connection with the plaza of the Queens Midtown Tunnel (see Fig. 10).

Another pertinent fact to consider is that traffic bound for a crossing may be fed into its approach streets via the immediately nearest intersection, controlled by a traffic light, from both intersecting streets, thus obtaining the benefit of practically the entire traffic-light cycle. On the other hand, traffic flowing away from the crossing can clear the next immediate light-controlled intersection only out of one of the two intersecting streets, and can thus utilize only part of the traffic-light cycle (assuming that right-hand turns are not permitted on the red light). Therefore, especially where only the smaller portion of the light cycle is available to diffuse the crossing's traffic, a special examination must be made of those streets to make certain that, in the aggregate, as well as in each of the four directions from the plaza, they have sufficient margins of capacity, above local traffic requirements, to carry peak crossing traffic without limiting the capacity of the exit lanes.

In addition to local traffic, parts of certain streets may be called upon to handle both entrance and exit traffic of the proposed crossing. It is essential to examine the probable peak traffic which they may be called upon to handle, and their adequacy to do so.

In congested business areas, furthermore, part of the crossing's traffic may approach the plaza from directions opposite to those of its points of origin. Thus, part of the passenger-car traffic may use an express shore drive, even at the expense of a "back-haul," to avoid the center-of-town congestion, and approach the crossing "from behind" its plazas. Truck traffic from waterfront terminal facilities may also approach the crossing in this way. Where such conditions are encountered, adequate convenient connections to the waterfront areas and highways must be provided.

These and similar considerations usually require that standard estimates be prepared of: (*a*) Peak-hourly traffic on approach streets in every direction to and from the proposed crossing plazas; (*b*) gross-hourly capacities of each of the approach streets; (*c*) their present and future peak-hour local traffic; (*d*) their margins and capacity available for the traffic to and from the proposed crossing; and (*e*) the probable peak-hour surplus margins (or deficits) over the traffic requirements of the proposed crossing (see Fig. 10).

Furthermore, for the purpose of appraising the relative merits, from a traffic-capacity standpoint, of the many different designs of plaza and approach connections which are usually prepared and studied before the final design is adopted, a series of traffic flows corresponding to each design must also be prepared. With the aid of these traffic flows, it is possible to weigh alternative designs on a quantitative basis, and eventually to select a plaza design and an adequate system of street approaches, acceptable not only from a traffic but also from other equally essential points of view.

NECESSITY FOR AND DIFFICULTIES ENCOUNTERED IN CHECKING TRAFFIC ESTIMATES

Realized Versus Anticipated Traffic.—In connection with the planning of free highway facilities, traffic estimates, if prepared at all, are seldom checked against traffic realized, once the facility is open. It is more than likely that such estimates would usually be exceeded, since the new highway would be so far superior to those existing and, costing nothing to use, would attract and create large volumes of traffic. In the case of vehicular toll facilities, however, traffic may be larger or smaller than estimated. In either event, a check-up of estimated versus realized traffic would be necessary.

If traffic volumes are greater than anticipated, they are often so much larger that a very substantial percentage of the capacity of the new facility is absorbed at once. If traffic volumes remain at high levels, plans for the expansion of the new facilities or the construction of new facilities are soon begun. If the traffic is smaller than anticipated, check-ups are necessary to determine the causes for the failure to attract the traffic forecasted, and to stimulate patronage of the new facility through advertising programs and proper "signing" of the highway routes leading to and from the new facility.

Total Traffic "Reservoir" Versus Divertible Shares.—In checking estimates versus realized traffic, it is generally found that there is least divergence in the data for the total traffic in the "reservoir" (see Fig. 6); but that fact is only of academic interest. On the other hand, the real differences occur in the estimated distribution of traffic among the competitive facilities; and that is a practical matter. It is not enough that the total traffic estimated in the "reservoir" be there—it is also essential that it be on the facility that was financed and constructed to attract the volume estimated. It is necessary, therefore, to explain differences in the distribution of traffic among competitive facilities and suggest means for attaining the distribution estimated.

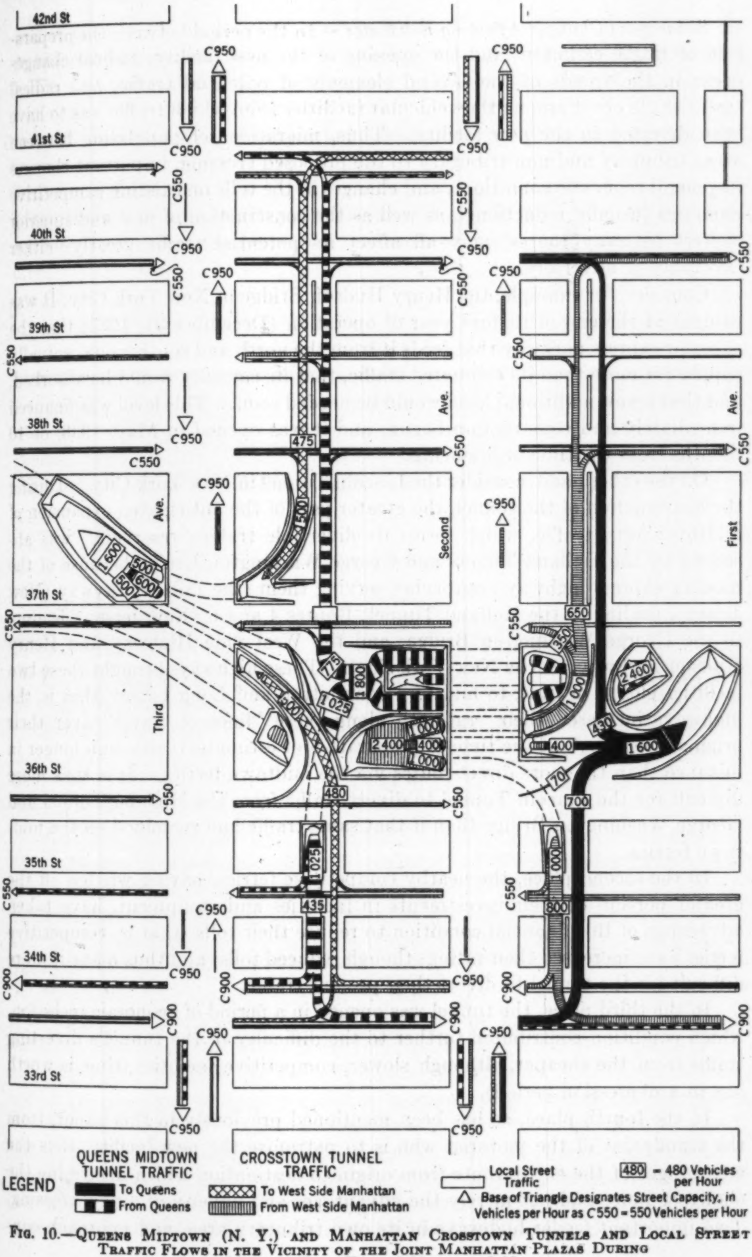
42nd St
41st St
40th St
39th St
38th St
37th St
36th St
35th St
34th St
33rd St
Third Ave.
Second Ave.
First Ave.
C950
C550
C900
475
500
1 600
500
600
500
775
1 800
1 025
200
2 400
1 000
400
1 000
480
1 025
435
650
350
1 000
2 400
430
400
1 600
70
700
1 000
800
LEGEND
QUEENS MIDTOWN TUNNEL TRAFFIC
To Queens
From Queens
CROSSTOWN TUNNEL TRAFFIC
To West Side Manhattan
From West Side Manhattan
Local Street Traffic
480 = 480 Vehicles per Hour
Base of Triangle Designates Street Capacity, in Vehicles per Hour as C550 = 550 Vehicles per Hour

FIG. 10.—QUEENS MIDTOWN (N. Y.) AND MANHATTAN CROSSTOWN TUNNELS AND LOCAL STREET TRAFFIC FLOWS IN THE VICINITY OF THE JOINT MANHATTAN PLAZAS DURING THE MORNING PEAK HOUR, 8:00 A.M. TO 9:00 A.M.

Subsequent Changes Affecting Estimates.—In the period between the preparation of traffic estimates and the opening of the new facility, radical changes occur in the trends of the several elements of potential traffic, and radical traffic shifts occur among the vehicular facilities from which traffic was to have been diverted to the new facility. Thus, migrations of population between areas tributary and non-tributary to the proposed crossing, important changes in general economic conditions, and changes in the tolls on existing competitive crossings (usually reductions), as well as the construction of new and superior express highway routes, may all affect its potential traffic greatly—either favorably or adversely.

Consider, for example, the Henry Hudson Bridge in New York City: It was evident at the end of its first year of operation (December 31, 1937) that the superior express parkway that feeds it from the north and south would actually supply far more than its estimated traffic, that its capacity would be absorbed, and that a new additional level would be needed soon. This level was financed immediately and construction begun, and it was opened in May, 1938, or 10 months from the time of financing.

On the other hand, consider the Lincoln Tunnel in New York City. During the construction of the tunnel, the greater part of the anticipated expansion of midtown ferry traffic, which forms its divertible traffic "reservoir," was absorbed by the Holland Tunnel and George Washington Bridge because of the modern express highway approaches serving them (the Pulaski Skyway (New Jersey), leading to the Holland Tunnel; Routes 4 and 6 (New Jersey), leading to the George Washington Bridge; and the West Side Highway and Henry Hudson Parkway, on the New York side). These highways brought these two facilities closer, in time, to midtown Manhattan and Long Island (that is, the highways increased their "ratings" along these "lines of travel" over their original "ratings," at the time of preparation of estimates), although longer in distance than the more direct routes via the midtown ferries. It is thus more difficult for the Lincoln Tunnel to divert traffic from the Holland Tunnel and George Washington Bridge than if that same traffic had remained on the midtown ferries.

In the second place, the nearby competitive ferries, having written off the greater portion of their investments in facilities and equipment, have taken advantage of this financial condition to reduce their tolls (that is, competitive ferries have increased their ratings though reduced tolls) and thus made it more difficult for the tunnel to divert their present traffic.

In the third place, the tunnel was opened in a period of economic recession, which condition contributed further to the difficulty of the tunnel's diverting traffic from the cheaper, although slower, competitive facilities (time is worth less in a depression period).

In the fourth place, as has been mentioned previously in this paper, from the standpoint of the motorist who is to patronize the new facility, it is the advantages of the entire route from origin to destination which determine the choice of route and not merely the advantages and novelty of a new crossing. Two important feeder highways in its own tributary area, and approach connections thereto, were incomplete when the tunnel was opened, whereas traffic estimates had been predicated on their completion. Consequently, its realized traffic naturally fell short of estimates. This lack of approach connections was remedied partly by the opening of the express approach connections through Union City, N. J., on June 30, 1939. As motorists become aware of these connections, there will be a shift from alternate routes to the tunnel. That such shifts are now in progress is strikingly illustrated in Table 10, which indicates that, as the tunnel's supporting highways come up to those contemplated in original estimates, its realized traffic gradually approaches original estimates.

TABLE 10.—PROGRESS OF REALIZED LINCOLN TUNNEL[a] VEHICULAR TRAFFIC TOWARD TRAFFIC ESTIMATES

Date	Description	Monthly traffic
May, 1938	Spring month close to average month of year; approach connections still under construction	151,653
October, 1938	Fall month close to average month of year; "loop approach" in Weehawken, N. J., opened to traffic October 15, 1938	172,154
May, 1939	Service streets paralleling express highway approach under construction opened in stages	203,046
October, 1939	Express highway through Union City, N. J., opened June 30, 1939	311,648
December, 1939		302,260[b]
Average month, 1939	As originally estimated by methods described in this paper	437,500

[a] Opened to traffic December 22, 1937. [b] Seasonal drop.

Thus, it will be seen that, in connection with the operation of new toll crossings, it is necessary to analyze, currently, traffic volumes of the entire "reservoir," segregated as to the sources of potential traffic of the new facility on a number of different bases, such as: By competing crossings, by days of the week, by types of vehicles, and by areas of origin or destination. It is also necessary to follow such changes as reductions in tolls of competing crossings; construction of new highways, particularly in areas immediately tributary to the crossing; changes in the economic situation and in parking regulations; and a host of other new, spasmodic, but related factors that affect the current volumes of the new crossing. All of these data are necessary in order to explain and to suggest means of meeting the changes which had taken place subsequent to the preparation of the original estimates.

Measuring Changes in Geographical Distribution.—Where current annual traffic data are compiled for all alternative routes, it is possible to check, at once, the changes in the total annual volumes that have occurred on competing alternate routes between the time of preparation of the estimates and the opening of the facility. It is more difficult, however, to determine the differences in the trends of traffic volumes in areas tributary, as compared with those not tributary, to the proposed crossing—especially since most crossings serve both types of areas. Traffic volumes over a period of years for individual areas practically do not exist. Fortunately, however, in connection with studies of the George Washington Bridge and the Lincoln Tunnel, and with the aid of

WPA personnel in recent years, origin and destination data were collected and compiled in 1925, 1930, and 1935. These data permit one to obtain a glimpse of the radically different rates of expansion that occur in traffic volumes originating in different counties, some tributary, and some not tributary, to the new facility (see Fig. 5).

Measuring Actual Diversions.—Moreover, traffic volume data segregated as between passenger cars, trucks, and buses are rare, and only in recent years have sample traffic counts permitted such figures to be compiled by The Port of New York Authority. Where current traffic volume data for crossings competing with the new one are available, it is possible, by a simple comparison of the volumes before and after the opening, to determined the traffic decline on these crossings. However, such traffic declines may be the result not only of (*a*) diversions to the new crossing, but of (*b*) shrinkages due to an economic depression or recession, or (*c*) shifts to facilities other than the new one, which have put toll reductions into effect, and (*d*) gains offsetting diversions stimulated by toll reductions and other traffic stimulating factors. It thus becomes practically impossible, at times, to segregate diversion losses from other concurrent losses, especially where successive changes in the policies of the operators of competing crossings make for radical shifts back and forth among competitive routes.

Distinguishing Current Fluctuations from Permanent Trends.—Thus, the conditions that arise upon the opening of the new facility are not only radically different from those upon which diversion estimates were predicated, but the current changes themselves make it exceedingly difficult, at times, to put the estimating methods to a practical test until a year or two have elapsed since the opening of the facility. At this time, other fundamental changes have taken place which must be considered. Looking back, these shifts may be ascertained more accurately but, for the operators, "post mortems" are cold and academic.

"Generations" Versus Diversions.—It is also interesting to make this observation: Although the anticipated traffic volume to be "generated" is invariably a highly intangible item, and although it usually forms a substantial share of the total potential traffic of a proposed crossing, this element, nevertheless, has been realized to a larger extent than other elements of potential traffic, such as diversions from competitive routes, which appear to be more tangible, although apparently more elusive in their realization. It is fairly definite, however, that new facilities, which make a contribution to speed, convenience, and safety of travel, can usually count on a substantial volume of "invisible" traffic ready to avail itself of the advantages they offer.

Checking Estimated Directional Traffic Flows.—Estimates made of the directional distribution of the traffic of the new facility, on its plazas and along its approach connections to and from the four points of the compass, can usually be checked only in a very limited area immediately adjacent to the new crossings. Beyond those points, local traffic usually merges with that of the new facility, and it becomes exceedingly difficult to differentiate between crossing and local traffic. Consequently, if congestion should occur in the vicinity of the new crossing, it becomes difficult to determine to what extent it is caused by the traffic of the new facility, and to what extent by the local traffic, which use the approach connections jointly.

These are only a few of the difficulties that one encounters in comparing traffic estimates with traffic realized in an effort to determine from past experiences the various and devious pitfalls into which the traffic estimator may fall, and to determine ways and means of realizing the traffic which the estimating methods discussed may indicate.

CONCLUSIONS

To those who have occasion to prepare traffic and revenue estimates of proposed self-liquidating vehicular toll facilities, the summary of the determining factors shown in Fig. 11, and the following directions, may be helpful.

Traffic Estimates a Balanced Appraisal of Economic Soundness.—The ultimate function of traffic and revenue estimates is to furnish the executive with a nicely balanced appraisal of the economic soundness of the proposed vehicular toll crossing. If the crossing is found to be economically practicable, the supporting estimates must be such as to inspire confidence that motorists will patronize the facility in sufficient volume to preclude financial embarrassment of the facility when it opens, and also, incidentally, prevent other communities from financing meritorious projects. If the proposed facility is proved economically unsound, the supporting estimates must be such as to assure the executive that the community is not being deprived of a facility it can well afford and would patronize, and that a loan is not being rejected on a project that may be a good financial risk. In short, estimates must be such as to aid the executive in deciding whether the motoring public shall enjoy a new traffic facility, or whether the investing public shall be saved the grief of an unsound investment.

It is obvious that traffic and revenue estimates that are too optimistic must be avoided. On the other hand, ultraconservative estimates are equally undesirable. Thus, if traffic and revenue estimates of a proposed facility are so conservative as to contemplate no expansion in the future, for example, they may be interpreted in two ways:

(1) The facility has reached its zenith and must inevitably enter a state of decline—in which case, why should the banker underwrite it? Or

(2) The task of peering into the future is too hazardous for the engineer—in which case the banker must set the price of the funds high enough to cover the heavy risk, thereby probably destroying its chance of being financed.

If engineers are to benefit from the construction of vehicular toll facilities that are sufficiently essential to the traveling public to be practicable economically, they must be willing to give serious thought to the economic as well as the engineering phases of vehicular facilities. Proof of the economic practicability of these facilities constitutes the inertia to their initiation. On the other hand, their financial success "generates" the demand for other similar

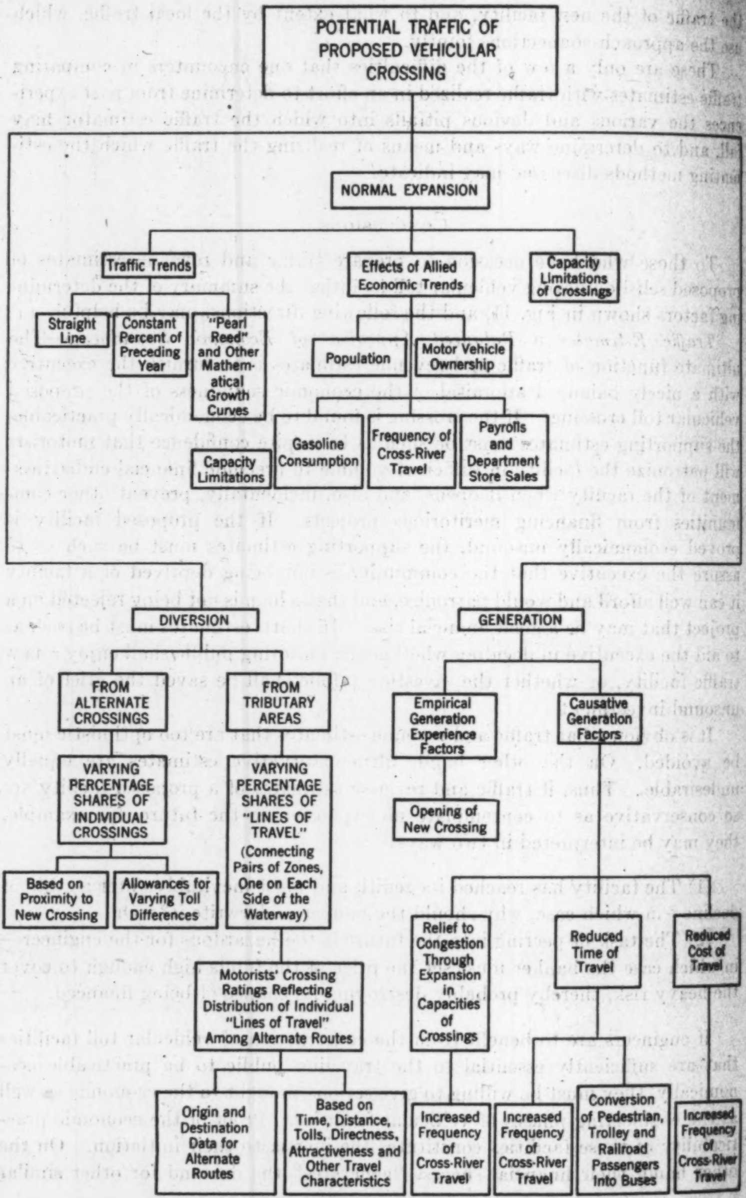
POTENTIAL TRAFFIC OF PROPOSED VEHICULAR CROSSING
NORMAL EXPANSION
Traffic Trends
Effects of Allied Economic Trends
Capacity Limitations of Crossings
Straight Line
Constant Percent of Preceding Year
"Pearl Reed" and Other Mathematical Growth Curves
Population
Motor Vehicle Ownership
Gasoline Consumption
Frequency of Cross-River Travel
Payrolls and Department Store Sales
Capacity Limitations
DIVERSION
GENERATION
FROM ALTERNATE CROSSINGS
FROM TRIBUTARY AREAS
Empirical Generation Experience Factors
Causative Generation Factors
VARYING PERCENTAGE SHARES OF INDIVIDUAL CROSSINGS
VARYING PERCENTAGE SHARES OF "LINES OF TRAVEL" (Connecting Pairs of Zones, One on Each Side of the Waterway)
Opening of New Crossing
Based on Proximity to New Crossing
Allowances for Varying Toll Differences
Relief to Congestion Through Expansion in Capacities of Crossings
Reduced Time of Travel
Reduced Cost of Travel
Motorists' Crossing Ratings Reflecting Distribution of Individual "Lines of Travel" Among Alternate Routes
Origin and Destination Data for Alternate Routes
Based on Time, Distance, Tolls, Directness, Attractiveness and Other Travel Characteristics
Increased Frequency of Cross-River Travel
Increased Frequency of Cross-River Travel
Conversion of Pedestrian, Trolley and Railroad Passengers into Buses
Increased Frequency of Cross-River Travel

Fig. 11.—Outline of Procedure for a Traffic Survey

facilities. The engineer has made, and can continue to make, two effective contributions to promote the construction of vehicular toll crossings:

(1) He must design facilities which, not only from an engineering standpoint but especially from a traffic standpoint, will be so far superior to existing competitive highway facilities that they will attract to themselves the bulk of the motorist patronage traveling between their tributary areas, as well as create new patronage, thus assuring that such projects will be financial as well as engineering successes; and

(2) Although the bulk of the traffic of the proposed toll crossing may not be presently "visible," he must discover such sound methods of measuring its potential traffic as will inspire the bankers' confidence whenever his prognostications point to its economic soundness and financial success.

ACKNOWLEDGMENT

The writer is indebted to The Port of New York Authority for the privilege of using the wealth of data collected in connection with its studies of the economic practicability of its vehicular crossings; to the WPA (Project No. 665-97-3-65) for assistance in the collection, compilation, and analysis of supplementary data; and to S. Woolman, assistant engineer, WPA, for aid in analyzing the basic data.

APPENDIX

MATHEMATICAL FORMULAS FOR DEVELOPING ESTIMATES OF DIVERSIONS, "GENERATIONS," AND EXPANSIONS IN TRAFFIC VOLUMES HANDLED BY VEHICULAR TRAFFIC FACILITIES

The following formulas describe, mathematically, the basic factors that affect, and the manner in which they bring about, changes in traffic volume. These formulas indicate the type of motorist mass reaction to changes in the underlying factors that affect traffic volumes. The values of the "constants" in these formulas must be determined, in every specific case, on the basis of data collected for the express purpose of determining these "constants" for the specific location and year.

The estimates resulting from the use of these formulas are "first approximations," rather than final estimates, and should be used only as aids to engineering judgment. Engineering judgment may point to the existence of "preference" or "prejudice" factors which are not measurable, but which are nevertheless real, and must be inserted in the formulas as additional factors likely to reduce or augment the volumes determined as "first approximations." The formulas are such as to allow for the insertion of judgment factors.

NOTATION

The letter symbols used in the formulas of this appendix are defined as follows:

a = a constant annual increment of traffic growth (Eq. 6);

C = cost; weighted average, one-way cost of a trip across the river (considering travel-time, distance, tolls and other route characteristics) along any given "line of travel," via all alternate crossings:

- C_0 = base cost (Eq. 11);
- C_1 = cost before the opening of a new crossing (Eq. 11);
- C_2 = cost after the opening of a new crossing (Eq. 11);
- ΔC = net travel cost difference between the proposed crossing and the "standard" crossing (Eq. 7);
- ΔC_d = monetary evaluation of the mileage difference, in cents per mile (Eq. 7);
- ΔC_r = monetary evaluation of the running-time difference, in cents per minute (Eq. 7);
- ΔC_w = monetary evaluation of the waiting-time difference, in cents per minute (Eq. 7);
- ΔC_p = "Prejudice" or "preference" cost difference of the convenience, attractiveness, directness, scenic values, and other directly immeasurable factors, in cents (Eq. 7).

ΔD = distance difference via crossing, compared to that via the "standard" crossing, in miles (Eq. 7);

d = discount factor by which unity (1) is reduced for every cent of excess travel cost over that via the "standard" crossing (Eq. 6);

F = frequency of travel, in one-way, cross-river trips per year:

- F_0 = frequency at a base cost of travel (Eq. 11);
- F_1 = frequency before the new crossing is opened (Eq. 11);
- F_2 = frequency after the new crossing is opened (Eq. 11);
- ΔF = percentage change in travel frequency, corresponding to a change in travel cost, following the opening of the new crossing (Eq. 11);

m = ratio of the margin of capacity to the annual volume in the base year (Eq. 10);

N = number of years between the base year and any given year (Eqs. 8, 9 and 10);

p = percentage of traffic along a "line of travel" handled by a given crossing (Eqs. 3 to 5);

q = premium factor; a percentage by which travel frequency is increased for every cent of reduction in the average trip cost (Eqs. 11);

R = annual rate of change in the ratio of margin of capacity, m (Eq. 10);

r = estimated "rating" of any proposed crossing (or any existing crossing whose factor ΔC has changed), based on its travel-cost differences from the standard competitive crossing; $r_1, r_2, \cdots r_n$, = the ratings of crossings $1, 2 \cdots n$; the rating of a "standard" crossing is unity (1);

ΔT = toll difference via any crossing compared to that of a "standard" crossing, in cents (Eq. 7);

t = time, in minutes:

Δt_r = running-time difference via any crossing compared to that via the "standard" crossing (Eq. 7);

Δt_w = waiting-time difference via any crossing compared to that via the "standard" crossing (Eq. 7);

V = annual volume of traffic:

V_0 = annual volume in the base year (Eqs. 8 and 9);

V_n = annual volume in any given year (Eqs. 8, 9, and 10);

V_m = maximum annual volume determined by the aggregate capacity of all facilities (Eq. 10);

ΔV = annual percentage traffic increase over preceding year;

v = the volume of traffic along a "line of travel" handled by all competing crossings (Eqs. 1 and 2):

v_n = volume handled by crossing (n) (Eqs. 1 to 5);

v_p = volume handled by the "standard" crossing (p);

$v_1, v_2, \cdots v_n$ = volume handled by crossings 1, 2, $\cdots$ n.

PERCENTAGE SHARES AND "RATINGS" OF EXISTING CROSSINGS FOR ANY GIVEN "LINE OF TRAVEL" BASED ON THE VOLUMES HANDLED BY EACH CROSSING

By definition (see "Notation"):

$$p_n = \frac{v_n}{v} \quad \text{(1)}$$

but

$$v = v_p + v_1 + v_2 + \cdots v_n \quad \text{(2)}$$

Therefore,

$$p_n = \frac{v_n}{v_p + v_1 + v_2 + \cdots v_n} \quad \text{(3)}$$

In Eq. 3 dividing numerator and denominator by v_p:

$$p_n = \frac{\frac{v_n}{v_p}}{\frac{v_n}{v_p} + \frac{v_1}{v_p} + \frac{v_2}{v_p} + \cdots \frac{v_n}{v_p}} \quad \text{(4)}$$

By definition, $r_n = \frac{v_n}{v_p}$; $r_1 = \frac{v_1}{v_p}$; $r_2 = \frac{v_2}{v_p}$; $\cdots r_n = \frac{v_n}{v_p}$. Therefore,

$$p_n = \frac{r_n}{1 + r_1 + r_2 + \cdots r_n} \quad \text{(5)}$$

"RATING" OF A PROPOSED CROSSING BASED ON COST DIFFERENCES FROM "STANDARD" COMPETITIVE CROSSINGS

Referring to "Notation":

$$r = (1 - d)^{\Delta C} \quad \text{(6a)}$$

or, approximately,

$$r = 1 - \Delta C\, d \qquad (6b)$$

EVALUATION OF TRAVEL CHARACTERISTICS OF THE PROPOSED CROSSING

Referring to "Notation":

$$\Delta C = \Delta t_r\, \Delta C_r + \Delta t_w\, \Delta C_w + \Delta D\, \Delta C_d + \Delta T + \Delta C_p \qquad (7)$$

MATHEMATICAL EXPRESSIONS OF TRAFFIC GROWTH

Growth consisting of constant annual numerical increments (straight line) may be determined by:

$$V_n = V_0 + a\, N \qquad (8)$$

Growth consisting of constant annual percentage increments (logarithmic straight line or compound interest curve) may be determined by:

$$V_n = V_0\, (1 + \Delta V)^N \qquad (9a)$$

or

$$\log V = \log V_0 + N \log\, (1 + \Delta V) \qquad (9b)$$

Growth consisting of variable annual increments dependent on available margin of capacity ("Pearl Reed" curve) may be determined by:

$$V_n = \frac{V_m}{1 + m\, R^N} \qquad (10)$$

EXPRESSIONS FOR CHANGES IN TRAVEL FREQUENCY FOLLOWING THE OPENING OF A NEW CROSSING OR NEW HIGHWAY FACILITIES

Referring to "Notation":

$$F_1 = F_0\, (1 + q)^{C_0 - C_1} \qquad (11a)$$

$$F_2 = F_0\, (1 + q)^{C_0 - C_2} \qquad (11b)$$

and

$$\frac{F_2}{F_1} = \Delta F = (1 + q)^{C_1 - C_2} \qquad (11c)$$

DISCUSSION

MURRAY D. VAN WAGONER,[5] M. AM. SOC. C. E. (by letter).—To attempt the solution of any great problem, whether it be in science, economics, politics, engineering, or the art of war, one must have a plan of attack. The paper by Mr. Cherniack furnishes the engineer with an excellent outline of attack upon the problem of determining the economic feasibility of a proposed vehicular crossing. The major difficulties to be encountered are mentioned and ways of overcoming them suggested.

The first step is to determine the boundaries of the territory from which the traffic for the new crossing will come. Then the amount of traffic in this "reservoir" must be determined. To determine this volume, short-time traffic counts must be made at the proper time of day, week, and year. It appears that the 12-hr period from 7 a.m. to 7 p.m. on Wednesdays or Thursdays, or both, with a supplementary count on Sundays in the months of April, May, September, or October (or in all four months), is the best time for such counts.

Knowing the contents of the traffic "reservoir," the portion of it likely to use a crossing such as the one proposed must be determined. This will be difficult and may be very expensive, but it is the most crucial point in arriving at a financially successful solution to the problem; in fact, it is the problem. Regardless of the fact that the traffic of the new crossing may be expected to come from both existing and new traffic, the existing portion is all that is available for study; and estimates of the portion to come from each must be based on what now exists.

A study of diversion experience factors of existing toll crossings shows such wide variation that only general conclusions can be drawn from them. They do indicate that the largest portions of the traffic of a new crossing will come from crossings nearest it and from those with "the smallest favorable toll spreads." To measure probable diversion accurately, origin-and-destination studies should be made and "lines of travel" established.

Mr. Cherniack gives a very novel method for estimating the probable diversion. For each competitive crossing a "relative merit rating" is established by giving consideration to waiting time, running time, distance, and amount of toll for the various "lines of travel" using it. Monetary value is assigned to these items and the final rating of each existing crossing determined by using this cost factor and the present volume of traffic carried. Then to determine the rating of the proposed crossing another equation is given. With the ratings of the existing crossings and that of the proposed crossing, the amount of divertible traffic is readily calculated. All this can be illustrated by simple mathematical equations.

A possible weak spot in the paper may be found in the equation for determining the rating of the proposed crossing. Mr. Cherniack does not explain just why this should be in the nature of a compound interest or discount formula; nor does he explain how the value of *d*, the amount of discount, is to be determined. It is merely defined as the "discount factor by which unity (1) is reduced for every cent of excess travel cost over that via the 'standard' crossing." He does not make the statement that the rating of the proposed crossing was obtained by using judgment and the equations. Now if one's judgment is to enter into the determination of a factor or term in a mathematical equation, the equation becomes merely a group of symbols representing an idea and ceases to be a mathematical tool.

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[5] State Highway Commr., State Highway Dept., Lansing, Mich.

It was found that comparative cost and comparative patronage of several existing crossings were related, but there was evidence of still other factors influencing the use of one crossing in preference to another. The conclusion to this was that there were certain things not easily measured in dollars and cents, such as scenery, safety, travel habits, etc., which play a part in a motorist's decision of which crossing he will use.

In the event that the divertible traffic appears insufficient to warrant the construction of a new crossing, one can (1) wait until it is sufficient, or (2) anticipate expansion. The first choice may result in abandoning construction of the crossing. The argument is then presented that if the new crossing is to be built merely to accommodate present traffic more advantageously, and no expansion is in sight, there is little use of building it at all. If the engineer builds with an eye not only to diverting some of the existing traffic to the new crossing but to future expansion of traffic, he must try to estimate the amount of this expected traffic.

There are all kinds of methods for estimating future traffic. A large variety of mathematical formulas has been used on such things as vehicle registration, population, gasoline consumption, etc., to estimate future traffic. All these have their statistical pitfalls. If the new crossing can add greatly to convenience of travel, new traffic will be generated automatically in addition to that which comes about by normal expansion.

Other structures fundamental to the success of a proposed toll crossing are adequate feeders, approaches, and plazas. Not only must these be adequate, but they should be superior to those servicing the competitive crossings; and, if they are not, they should be made so, at the same time taking care that they do not also aid competitive crossings. It is interesting to note the increase in traffic through the Lincoln Tunnel as various streets leading to it are improved.

It may also be necessary to conduct advertising campaigns to increase patronage. According to past experience, the lowering of tolls does not usually increase revenue through increased traffic.

The importance of checking traffic estimates must not be overlooked. It is quite possible that between the time of the original estimates and the completion of the new crossing radical changes such as economic depressions, increased facilities for feeding competitive crossings, toll rates, etc., may take place and these materially affect the traffic over the new crossing.

Charles B. Winick,[6] **Assoc. M. Am. Soc. C. E. (by letter).**—An able and logical exposition of the elements entering into the preparation of a survey which deals with the prediction of traffic for a newly proposed vehicular crossing is presented in this paper. It is a source of gratification to know that even so complicated a subject as traffic analysis, with all its accompanying ramifications, can be brought to some semblance of logical conclusions in the hands of a competent observer. The numerous factors, technical as well as otherwise, are so complex in a study of such character that one must be constantly on the alert in order to avoid the pitfalls of unsound and illogical conclusions which are likely to be drawn unless the accumulated data are thoroughly scrutinized and examined minutely, all their elements segregated, and their relations to the entire subject carefully studied.

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A little reflection will reveal at once the importance of traffic analysis to the projection of a new vehicular crossing. Proper traffic analysis is the most important tool in the hands of the engineer, banker, industrialist, and politician before any proposed vehicular crossing may be either recommended or entirely rejected. Even after the project is recommended its exact location, kinds of approaches, connecting highways, future provisions for expansion, etc., must all be studied in the light of traffic analysis. Even the alinement and grade of the proposed crossing must be determined for the convenience of the motorist who is to use the facility and who in the last analysis is the initial unit of any traffic study.

It is incumbent upon the engineer charged with the responsibility for the construction of a large project to prepare all the necessary data in order to arrive at a proper recommendation as to whether such project is physically and economically sound, and what advantages, if any, it may have compared to another type of facility—say a bridge versus an underground tunnel. Will it be self-sustaining? Will it produce an unnecessary economic burden on the communities that are to enjoy the benefits of such facility, or will it enhance economic life? Will the architectural treatment of the facility become a ban or a boost? Will esthetic values be modified? Many other tangible and intangible factors must be known prior to the time the engineer is ready to argue his case meritoriously before any public body or financial institution which has an interest in the creation of such facility.

Once the type of crossing and its location are determined it becomes a relatively easy task to estimate within reasonable limits the cost of construction, cost of real-estate acquisition, administrative and engineering costs, etc., which enter into the total estimate of the project cost. After computing the operating and maintenance charges, the cost of amortization, interest, taxes, etc., the engineer is generally in an advantageous position to know, more or less with exactness, the amount of annual revenue required in order to meet all charges during the useful life of such project. The difficult task of determining the toll charge becomes the most important factor that will determine the feasibility of the project.

If the toll is predetermined (as is most often the case), the facility must be so designed and planned as to attract the required amount of traffic corresponding to the total required revenue. In cases where the toll charge is determined because of other free or competing facilities, or any other considerations, the most accurate traffic analysis will of necessity become prejudicial, since all interests connected with the promotion of the project will tend to develop some one or another thesis that the required amount of traffic will be met or surpassed. Since most traffic consists of diverted and generated components, both of an uncertain nature, particularly the "generated" portion, it can be seen readily that it is difficult for a conscientious and honest analyst to avoid the danger of being somewhat too optimistic; and for that reason an unbiased analysis by a competent observer disinterested in the project is always desirable.

Use of Empirical Formulas.—As is the case with any empirical formulas, the expressions of traffic growth are subject to criticism due to the variation of the several components that enter into making up such a formula. Whether the straight line, compound interest, or Pearl Reed curve is applied to estimate the annual traffic growth, accuracy is sacrificed at the expense of the several assumptions that are physical in nature and subject to wide variations. Thus, in the straight-line formula the assumption that the annual increment of growth is constant is obviously erroneous. It takes but little observation to conclude that the annual increment of growth is dependent on too many physical changes to remain constant for any considerable period of years. Similar objections exist in any of the other empirical formulas expressing traffic growth.

Rating of a Proposed Crossing Based on Cost Differences.—Again, in endeavoring to obtain the merit rating of any proposed crossing relative to existing crossings in the same "line of travel," the basic assumption of cost differences, monetarily evaluated, is subject to numerous variations. Taking the case of $\Delta T_w \Delta C_w$, which represents the product of the waiting-time difference via any crossing compared to that via the "standard" crossing by the monetary evaluation of the waiting-time difference, in cents per minute, note the following:

The monetary cost of waiting time of a highly paid executive who is en route to negotiate an important business contract cannot be compared to that of the "young lover" who is leisurely taking out his girl friend for a joy ride. Time is of no essence to the latter and hangs heavily on the former. Although it is true that the average commuter's time, to and from work may be evaluated with some degree of accuracy, no such appraisal of the cost of time can be made for a patient who is hospital bound by ambulance for an emergency surgical operation which is a matter of life itself. The same considerations would also hold true of $\Delta T_r \Delta C_r$ representing the product of the running-time difference via any crossing compared to that via the "standard" by the monetary evaluation of the running-time difference in cents per minute. Many more examples can be cited, but the foregoing are sufficient to illustrate the differences arising in waiting-time or running-time costs between extreme cases; and therefore the application of a constant rate of cost for such factors to all types of motorists would obviously lead to some erroneous conclusion. It must always be remembered that a variation of any basic assumption in the rating formula affects the merit rating of the proposed facility, and students of traffic analysis should pause for a moment before accepting the author's recommendation of merit rating as a basis for predicting future traffic on a proposed crossing.

Preference and Prejudice Factors.—It is easy to draw a smooth curve between any two points and conclude that such curve is a function of the preference or prejudice factors of a given route along any given "line of travel." The contention that the "scatter" of the data points revealed by a plot of relative patronage against relative cost has any significance is questionable. Even if each motorist were stopped and questioned as to his reason for using a given crossing in preference to any other competitive crossing along the same line of travel, such data could not be applied to a forecast of probable traffic on a proposed new facility, for it does not follow that the same reasons under similar circumstances would motivate another motorist or even the same motorist to use the new facility.

Relative Patronage Factors.—Closer analysis of relative patronage factors would indicate that they are not only psychological and economic in nature but geographical and social as well. For instance, a facility built for the use of car owners representative of an agricultural or farming community would be affected quite differently from that of an urban community of the same population, largely because of the different habits and customs of the two types of communities. Again the social characteristics of a typical New England Yankee community, being entirely different from those of a community peopled by foreign-born elements, would result in marked variations of "relative patronage" for a given or newly proposed facility. Thus, it can be seen that it becomes extremely difficult to arrive at any rational figures which represent all the factors that contribute to relative patronage and relative cost data.

Trip Cost.—Were the average motorist to reflect a moment on the actual cost per trip that he ultimately pays for crossing a given facility, the engineer would be in an eminently qualified position to actually calculate the volume of traffic with reasonable precision. It would take much effort, however, to educate the average motorist to the actual cost per trip since he is not readily receptive to any assimilation of statistical data which directly affect him. Even if an educational campaign were successful, the motorist would soon become cost-conscious, and this would result in putting most of the automobile finance companies out of business, causing a material reduction in traffic growth.

The author assumes a cost of waiting-time difference at 2 cents per min and running-time difference at 3 cents per min. Such assumptions are more or less arbitrary since the patronage of any facility is entirely dependent on geographic, psychological, social, and economic factors as stated previously. A variation, for example, of the waiting-time difference to, say, 3 cents per min instead of 2 cents would dislodge all of the figures as shown in Table 6 and would produce different relative patronage distribution of each facility on a given "line of travel." Therefore, basing the forecasting of traffic for any new facility on such arbitrary assumptions may lead to erroneous conclusions. Thus, it can be seen that even a neutral observer, representing the motorist and taxpayer and not the banker, builder, or financier, is confronted with the most difficult task when endeavoring to evaluate all of the elements entering into any equation contributing to the patronage factors of any proposed facility.

Projection of Traffic Into the Future.—It cannot be denied that the number and value of the merit rating of competitive crossings act as a barometer for the traffic distribution of all components along a given "line of travel." Were the merit ratings of such nature that their determination depended on fixed and tangible elements, such a method would offer a ready means of projecting traffic into the future. Unfortunately such method is open to the numerous objections previously mentioned, and, even in the hands of an expert analyst who is quite familiar and able to modify such intangible concepts, a quantitative forecast may fall short of actual performance and often does, as was the case with the Lincoln Tunnel.

Nevertheless such analytical data are of considerable importance qualitatively, since thay enable one to predict trends and traffic patterns. Furthermore, when upon opening the new facility, comparisons of actual with predicted traffic performance are made, noted discrepancies can be studied and accounted for in the light of changes that occurred between the time that the traffic studies were made and the time of the actual opening of the facility. Suffice it to say that the improvement of any competing facility prior to opening the new facility would upset any prediction favoring the latter. A case in point, as the author states, is the completion of the West Side express highway which shortened the time between the Holland Tunnel and the George Washington Bridge and affected the predicted traffic for the Lincoln Tunnel adversely. It also can be seen at a glance that an improvement at the Manhattan approach to the Queensboro Bridge would affect the traffic materially on the Queens Midtown Tunnel. Although the Queensboro Bridge is heavily traveled, no appreciable delays are encountered on the span itself. The Manhattan approach could be modified and modernized by: (1) Removing the Second Avenue Elevated Line; (2) demolishing a substantial number of buildings; (3) creating a large plaza; (4) rearranging traffic signaling so as to permit traffic to proceed northward on Second Avenue; (5) building private streets where necessary; (6) eliminating or at least modifying the traffic light, or possibly reconstructing that part of the bridge which provides access to Welfare Island; etc. Under conditions of maximum improvements at both ends of the bridge the nearly "ideal" state of travel on such facility would be reached, and it could then be determined whether the bridge is overcapacitated or capable of absorbing additional traffic.

That a crossing is as good as ability of traffic to enter or leave it may be seen further from a survey of conditions of the East River bridges at the lower end of Manhattan. Those approaches were designed for "horse-and-buggy" days and are now entirely obsolete. Any improvement, therefore, on such approaches would soon be reflected in a traffic redistribution of all other crossings in the same "line of travel" relative to the ease and comfort, time saving, etc., that such improvement would afford. Thus the widening of Chrystie Street at the Manhattan end of the Manhattan Bridge has resulted in a marked increase of traffic on that bridge. It may be concluded that unless all competing facilities along a given "line of travel" are so constructed or altered as to receive traffic under ideal or nearly "ideal conditions any attempt to project traffic into the future" is vulnerable to a redistribution, commensurate with improvements of other competing crossings. Witness the hopeless congestion at Canal Street, in Manhattan, upon leaving the Manhattan Bridge. Surely a modification of that approach would have a marked effect on relative traffic patronage on any of the other bridges in that area. Again, the much talked of necessary improvement of the Brooklyn Bridge would affect a traffic redistribution on the lower bridges and any forecasting of traffic for the newly proposed Battery-Hamilton Avenue Tunnel (Manhattan to Brooklyn, N. Y.) would thereby become subject to considerable modifications and revisions.

Relative Toll Charges.—Prior to the fixation of a permanent toll charge for the purpose of raising sufficient revenue to insure economic practicability for the promotion of a contemplated crossing, several factors must be studied. After all, when the engineer compiles his estimate of the cost of construction, operation, maintenance charges, etc., he is reasonably certain as to how much gross income will be required in order to meet the necessary amortization, interest charges, taxes, cost of financing during construction, operating costs, maintenance cost, etc. Assuming an arbitrary toll charge, a certain minimum amount of traffic would be required to insure economic success of the project. Such amount of traffic is either in the proximity of the forecasted traffic as prepared from other characteristics or, if deficient, the toll must be increased to meet all the required charges. Once the arbitrarily assumed toll charge is increased an unbalancing of the forecasted traffic results. Such would be the case particularly when competing with free facilities. It thus may become a vicious cycle of increasing the toll and correspondingly decreasing the traffic that produces revenue. Thus, taking the case of either the Queens Midtown Tunnel or the proposed Battery-Hamilton Avenue Tunnel, it is safe to conclude that a toll charge of 10 cents per car would attract considerably more traffic than if the toll were, say, 25 cents or 50 cents per car. There is undoubtedly some point of balance at which the economic law of diminishing returns becomes operative. This point must be determined carefully in order to arrive at a maximum economic yield before any intelligent forecast of future traffic is finally determined. Such intangible assets as increased or decreased land valuation in the approach vicinity, industrial and commercial benefits, public convenience, etc., although difficult of economic evaluation, nevertheless affect markedly the determination of toll charges. The writer is of the opinion that a variable toll charge for different periods during the life of the facility is likely to result in a more uniform distribution of traffic than a charge fixed at the opening of the facility and remaining constant during its entire life.

Generated Traffic.—The writer is keenly interested in ascertaining whether there is any reliable, scientific method of forecasting the "generated traffic." Reliance on past experience and former observation is too uncertain, especially when it is realized how important a factor such an element may become. Frequently the introduction of a new river crossing at a certain location may result in a shift of the entire residential or industrial community at either one or both ends of the crossing, causing a corresponding change in traffic distribution generally and in the "generated traffic" particularly. Cases in point are the Holland Tunnel and, say, the Brooklyn Bridge. It was not so long ago that the area in the vicinity of the Manhattan approach to the Holland Tunnel was occupied by old, dilapidated, and unsightly buildings. No sooner had the tunnel been placed in operation than active construction of commercial, industrial, and even residential buildings not only transformed the slum and blighted areas in the approach vicinity into modern developments, but such activity necessarily resulted in a spurt of generated traffic. In contrast, the real estate in the "shadows" of the Brooklyn or Manhattan bridge approaches has depreciated constantly and property owners are forced to neglect maintenance and repairs. This causes obsolescence and unsightly ugliness, and is creating one of the worst slum areas in the city. In such cases the "generated" traffic is continuously declining. Under such extreme circumstances, how is any one to arrive at a rational method of predicting the annual increment of generated traffic?

It may very well be that more careful attention to esthetic values will result in better architectural treatment of future crossings, whether it be a bridge or a tunnel, and the resultant upheaval of real estate will be minimized. It is hoped that future approaches will be so treated as to enhance the property values of real estate, enabling the property owners partly to finance the cost of such improvement.

Traffic Determinants.—The determination of future traffic by such determinants as gasoline consumption, department store sales, payrolls, purchasing power, or what have you, is as good a method as is a forecast based on past performance; but even such determinants are subject to wide and unforeseen variation. It may very well occur that the new crossing as projected may serve an industry which may thrive, despite economic depressions or recessions. Department store sales, the total consumption of gas, and all such other determinants may take a decided drop when computed for the entire traffic reservoir. Nevertheless the traffic for the newly projected facility may still show an upward tendency; or the reverse may be true—that is, while all traffic determinants may increase, traffic for a single given facility may show a decrease. In short, it would seem unsafe to forecast traffic for a newly proposed facility on such determinants.

Analysis of Traffic on Streets in the Proximity of the Plazas.—The assertion that the increase of traffic on the city streets due to a newly constructed adjacent plaza from a crossing may be forecast with a reasonable degree of accuracy is open to the following objection. Experience will show that motorists accustomed to use a certain route when traversing the city, if unduly held up because of additional traffic from an adjacent plaza or if delayed on account of increased traffic from whatever cause, will soon abandon such route and take the adjacent streets paralleling the customarily traveled route. A new redistribution of traffic in the vicinity of the approach streets would soon occur, and the forecasted results for increased traffic for such streets due to the addition of the new facility would become entirely unbalanced. If the entire area in the vicinity of the new facility is already suffering from excessive traffic, it is only reasonable to conclude that provisions to absorb the additional traffic from the new facility must be made either in the streets immediately contiguous to the plazas or to adjacent parallel streets. Of course, a solution may also be found by improving traffic conditions on the existing streets by elimination of parking, alteration of signaling, or numerous other devices which will facilitate the movement of existing traffic.

In view of the nature and complexity of the subject the author's attempt to standardize and segregate all the elements affecting traffic is a worthy contribution to such a highly controversial subject. The mere recognition of all elements affecting traffic is a difficult task in itself. The author's logical treatment, simplicity and brevity of the theoretical analysis, and the various empirical formulas offered for use, as well as the ready applicability of the several methods suggested to the solution of practical problems, are worthy of high commendation. Mr. Cherniack is indeed to be congratulated for his effort to lighten the burden of engineers who are always in a dilemma when a new crossing is projected, lest traffic predictions may be insufficient to proceed with the planning of the new facility. The importance of such traffic data, even if not entirely quantitative, cannot be overemphasized. Undoubtedly other means will be found ultimately to further the accuracy of the author's proposed method for estimating traffic. Even new concepts and ideas may be developed, taking into account the possibility of changes in modes of traffic, modification of existing vehicles, road changes, urban and suburban population, traffic redistributions, effect of aerial travel development, etc. The author's development of the subject with the present meager data at his disposal should act as a stimulant to other engineers engaged in such fields, for it is only through the constant accumulation of more and more reliable data that traffic engineers may ultimately avert the many "traps" open to a study of such type. It is indeed a source of gratification to feel that the "right start" in the "right direction" has been made.

JAMES H. S. MELVILLE,[7] M. AM. SOC. C. E. (by letter).—Mr. Cherniack is to be congratulated on having written what is probably the first comprehensive paper on the nebulous art of estimating the probable traffic and revenue for a projected toll facility. His exhaustive review of the steps necessary for the formulation of a proper judgment on the part of the engineer in charge of such work has been ably presented and leaves little room for comment or criticism. Traffic engineers will welcome the fact that he has made generally available some of the valuable data on traffic and traffic habits collected over many years in the laboratory of The Port of New York Authority.

Although the majority of his data and the analyses drawn therefrom are directly applicable to the metropolitan area of New York City, and to traffic in that and other cities of a comparable nature, the principles enunciated and the conclusions drawn are applicable to any situation, provided the necessary data as to traffic volumes, origins and destinations, rates of growth, and relative costs of travel can be determined adequately for the location in question. The formulas developed by Mr. Cherniack in the Appendix of his paper are interesting and their reliability will doubtless be tested by application to specific instances in the future. Since one of the major problems in making estimates of prospective toll traffic is the correct interpretation of the available data pertaining to existing traffic and its habits, any mathematical formula which will give even a "first approximation" will be welcomed by traffic engineers as a concrete factor by which their own judgment can be checked and possibly corrected—especially when the formula is based on long and intelligent study of traffic habits and preferences in such a diversified area as the metropolitan district of New York City.

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Engineers engaged to report on the traffic and revenue possibilities of toll facilities are usually handicapped by the inadequacy of the time available for the making of proper traffic studies. As a general rule, three to four months is the maximum time in which field work, tabulations, analyses, and conclusions have to be completed. For an important project only one set of field observations can be made, tabulated, and analyzed in that period of time. Unless there are existing toll facilities in the locality from which seasonal traffic patterns can be obtained, this is too short a time in which to obtain them independently, and the lack of local data similar to that contained in Mr. Cherniack's paper makes it difficult to supplement field observations and to formulate reliable premises for the foundation of estimates. It is hoped that the publication of this paper will give an impetus to the publication of further studies in different localities by representatives of public agencies.

N. CHERNIACK,[8] ASSOC. M. AM. SOC. C. E. (by letter).—In his discussion of the paper, Commissioner Van Wagoner has labeled it properly an "outline of attack upon the problem of determining the economic feasibility of a proposed vehicular crossing." It was intended to be just that. He has presented an excellent brief outline of the paper and has "put his finger on a weak spot," which, at the same time, happens to be one of the most difficult phases of estimating potential traffic. That phase relates to: (1) The determination of the "relative merit ratings" of all alternate routes on the basis of the present volumes these routes now handle along a number of "lines of travel"; (2) estimating the corresponding "relative merit ratings" of the proposed route on the basis of its travel characteristics relative to its competitor routes; (3) from these "ratings" determining the redistribution of travel, assuming the new route to be opened; and (4) thus estimating the shares which the new route would handle of each "line of travel." Commissioner Van Wagoner also justifiably complains that the writer has not explained why the rating equation should be in the nature of a compound interest or discount formula, nor how the value of d, the amount of the discount, is to be determined.

No economic law that is to be expressed by a mathematical equation can ever be induced from the observed quantitative data. The data may indicate the general type and "family" of curves that would best express the law mathematically, but the specific mathematical function must be selected arbitrarily by the investigator. After selecting such an equation, the data then will permit the investigator also to determine the "constants" in the equations by the "least-square" or other analytical or graphic method. It might also be pertinent to mention that the "least-square" method is limited to the determination of "constants" in equations that can be converted into "straight-line" forms, by means of logarithmic, trigonometric, or other functions.

Thus, consider the particular problem at hand—that of establishing the law between the "relative merit rating" (or relative patronage) of a given route along a given "line of travel" and the several factors that motorists take into account in allotting their patronage—namely, running and waiting time, distance, tolls, and such other "preference" or "prejudice" factors as scenery

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(slums), safety (hazards), travel habits, etc. The simplest inverse relationship between the "relative rating" and the average inconvenience relative to the best crossing may be expressed by a straight line with a negative slope, as in the approximate Eq. 6*b*, in which the discount factor d is the constant slope in the straight line. Its "constants" may be determined as follows: Eqs. 6*b* and 7 are first combined into

$$r = 1 - \Delta t_r \, \Delta C_r \, d - \Delta t_w \, \Delta C_w \, d - \Delta D \, \Delta C_d \, d - \Delta T d - \Delta C_p \, d \ldots . (12)$$

Eq. 12 being in the "straight-line" form, its "constants" may now be obtained by the "least-square" method by setting up as many equations as the data will permit. In each equation substitute the observed values of Δt_r (difference in running time along the "line of travel" via the given crossing and relative to the best route), Δt_w (relative waiting time), ΔD (relative distance), and ΔT (toll difference). Now solve by "least squares" to determine $\Delta C_r \, d$, $\Delta C_w \, d$, $\Delta C_d \, d$, d, and ΔC_p. Having determined d, solve for ΔC_r (the value of the running time difference), ΔC_w (value of the waiting time difference), ΔC_d (value of the distance difference), and ΔC_p (the combined value of the "prejudice" and "preference" factors). Of course, ΔC_p may be broken up into several individual "preference" or "prejudice" factors applicable to specific sections of routes, some of which may be particularly attractive and some particularly repulsive to travel. These values of the "constants" would constitute "first approximations."

Analysis of a great many "lines of travel," however, indicated that even the poorest alternative route carries some competitive travel. Hence, the "rating" of even the poorest crossing is practically never zero, and from its nature it can have no minus value. Therefore, a straight line with negative slope is only a crude approximation of the "rating" equation. Some curvilinear relationship would be a more realistic expression. A hyperbola, for example, might have been chosen as one family of such curves. The writer, however, chose the discount or compound interest curve, since it is a familiar everyday concept in financial transactions and lends itself to simple application to this problem. It may be translated to mean that the average motorist "discounts" the value of the route in the same way that a realtor would discount the value of a parcel of real estate—for example, relative to some other parcel of known value on the basis of factors which he knows affect real-estate values.

If Eq. 6*a* is adopted, therefore, it can first be readily converted to a straight line, logarithmically, as follows:

$$\log r = \Delta C \log (1 - d) \ldots\ldots\ldots\ldots\ldots\ldots (13a)$$

Then let $r' = \log r$ and $d' = \log (1 - d)$, so that

$$r' = \Delta C \, d' \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13b)$$

Then by combining Eq. 13*b* with Eq. 7, the values of the "constants" may again be determined by the "least-square" method.

It will be seen that one's judgment need not enter into the determination of the values of the independent factors of costs of running time, waiting time, distance, and the values of the "prejudice" and "preference" factors or the discount factor "*d*," in the mathematical Eqs. 6*a*, 6*b*, and 7. These equations, therefore, are powerful tools in determining the values of these factors as placed by motorists in the aggregate at the time when, and the place where, the origin and destination surveys were made. On the other hand, the "constants" in these equations need not be determined strictly and rigorously by the "least-square" method. They may be determined by analytical, graphical, or trial-and-error methods, and even modified to any extent desired by the judgment of the investigator, arising from his intimate knowledge of conditions affecting specific routes. The equations as such, at the same time, picture clearly and accurately the manner in which motorists in the aggregate distribute their travel among routes of different attractiveness, as reflected by the several factors.

In his discussion of the paper, Mr. Winick was kind enough to state that the writer has at least rationalized the subject of traffic analysis. On the other hand, he proceeded at once to make a number of very serious criticisms as to the application of the principles presented, the most important of which, summarized, are that: (1) Future events are so much in the dark that the problem of measuring them is almost insurmountable; (2) the estimator of traffic and revenues of a proposed project is subject to greater pressure to raise estimates than the estimator of costs of construction to reduce costs; and (3) the "personal equation" is the predominant factor in estimating traffic and revenues.

In reply to the first criticism, it can be said that, on the one hand, the engineer is accustomed to taking account of future events, as in estimating future costs, for example. On the other hand, his projects also influence the future. In so far as the engineer controls a part of the future, he should be able to estimate its effects on his figures. Thus, it is now generally admitted that every new facility will "generate" some new traffic. In the past, however, engineers have "leaned over backwards" in estimating this item and, as a consequence, it has been invariably exceeded. In fact, in planning free highway facilities engineers may well be criticized for having either overlooked this item entirely, or having failed to give it sufficient value. The result has been that every new super-highway, by virtue of its tremendous superiority, has been swamped and clogged in the first few years of its life. For example, the writer is prepared to admit that traffic "generation" which results from the building of a vehicular project may be extremely difficult to measure. Its measurement, however, is not an insurmountable task by any means. It can be measured if sufficient data are collected and the necessary research to do it is properly conducted. As in other problems of research, the engineer has the choice of "fight or flight."

In his second general criticism, Mr. Winick states that

> "In cases where the toll charge is determined * * * the most accurate traffic analysis will of necessity become prejudicial since all interests connected with the promotion of the project will tend to develop some one or another thesis that the required amount of traffic will be met or surpassed."

In effect, Mr. Winick thus accuses the traffic analyst of dishonesty under pressure.

Usually, traffic and revenue estimates for any given vehicular project are prepared simultaneously with estimates of costs of construction, maintenance, and operation. Subsequently, revenue estimates are balanced against cost estimates to determine economic practicability. In fact, it is often possible to make several approximations of anticipated traffic and revenues much before the corresponding cost estimates are prepared. These "horse-back" estimates indicate roughly, of course, the approximate annual costs of the project that the anticipated revenues could sustain. Under those circumstances, the cost estimating engineer could not be accused of cutting his cost estimates "under pressure" to meet the revenue estimates in order to get the construction program started. Professional integrity must be taken as an axiom if the engineer is to obtain the confidence of the public and the financier.

The third serious point that Mr. Winick makes is that, in the formulas prepared by the writer, so much engineering judgment is required that there would be a wide variation in the results calculated by different engineers. Therefore, it would be "difficult for a conscientious and honest analyst to avoid the danger of being somewhat optimistic," according to Mr. Winick, who suggests that "for that reason an unbiased analysis by a competent observer disinterested in the project is always desirable."

An independent review of any project is always desirable. That is the proper function and the contribution of consulting engineers to any project, especially of any magnitude.

That the formulas developed by the writer involve too much of the "personal equation" is a statement unsupported by facts. If, with the aid of the formulas, Mr. Winick is fearful of too great an optimism, how much more optimism would develop without any guide whatever! In fact, the formulas were developed by the writer to present some genuine restraining influence on the traffic analyst that has heretofore been lacking. On more than one occasion the writer has seen the pitfall illustrated by the following simplified example:

Assume two bridges, *A* and *B*, say 5 miles apart, tapping similar areas, the roads and approaches leading to them being about the same, and each, therefore, handling about equal traffic—say 5,000,000 vehicles annually. Now assume a proposed bridge, *C*, midway between and about equal in traffic characteristics and capacity to bridges *A* and *B*. In the computation of divertible traffic, it was assumed that since the proposed bridge *C* was about equal trafficwise to bridge *A*, it would divert about half of *A*'s traffic. Also bridge *C* being equal trafficwise to bridge *B*, it would divert about half of *B*'s traffic. The divertible traffic of bridge *C* was thus estimated as 5,000,000 vehicles. This did not seem to be out of line, especially where the amount was integrated from the summation of a large number of "lines of travel." Under pressure, sometimes consulting engineers have not had sufficient time to check their conclusions to observe what traffic had remained on bridges *A* and *B*. Bridges *A* and *B*, of course, were left with 2,500,000 vehicles each, but bridge *C* was to handle 5,000,000 vehicles. Why? The three bridges are equally attractive. Should they not handle one third the existing traffic? ("Generation" is omitted at this point.)

By means of the rating formula developed by the writer, bridge *A* and bridge *B* each has a rating of 1 derived from its traffic volume. The share that each now carries is obtained either from its traffic directly or by dividing each rating by the sum of their ratings, each thus handling half of the total. Under the proposed conditions, the estimated rating of bridge *C*, being equal to either *A* or *B*, would also be 1. Its share would now be obtained by dividing its rating 1, by the sum of the ratings of the three bridges or 3, or a share of one third. The rating formula thus restrains honest optimism because it forces the engineer to abandon the concept that a new crossing merely diverts traffic from the existing crossings, to the concept that a new crossing produces a redistribution of traffic among the existing and new crossings. Furthermore, under the estimated redistribution, the formulas force the engineer to determine the traffic that each of the present crossings will be left with as well as the traffic that the new one will divert from the existing crossings. Under this concept, expressed by the rating formulas, reasonable and less optimistic judgment is obtained even without a quantitative application of the formulas. The purpose of any of the formulas proposed is not merely to substitute blindly the values of the "constants" and solve algebraically, but to give the engineer a quantitative concept of the phenomenon of distribution and redistribution of traffic flow under changed and changing conditions.

Table 11 shows a hypothetical case of traffic distribution under existing facilities and a redistribution under proposed conditions, assuming the new crossing *E* had the same rating as the best existing crossing *A*. The rating of existing crossings is assumed to be proportional to the volumes of traffic now carried along the given "line of travel" and the rating of the proposed crossing assumed to be based on its route characteristics.

TABLE 11.—HYPOTHETICAL CASE OF TRAFFIC DISTRIBUTION UNDER EXISTING FACILITIES AND A REDISTRIBUTION UNDER PROPOSED CONDITIONS

Crossings	UNDER EXISTING CONDITIONS			ASSUMING NEW CROSSINGS IN OPERATION		
	Annual vehicles (millions)	Percentage of total	Rating factor	Rating factor	Percentage of total	Annual vehicles (millions)
A	5.0	50	100	100	33.3	3.33
B	3.0	30	60	60	20.0	2.00
C	1.5	15	30	30	10.0	1.00
D	0.5	5	10	10	3.3	0.33
Old sum	10.0	100	200			
E	....	....	....	100	33.4	3.34
New sum	....	....	....	300	100.0	10.00

It will be noted that the traffic redistribution, assuming a new one is opened, is of the present volume of traffic. This computation involves no estimates of any future increases or decreases at this point, but only a redistribution.

Rating of a Proposed Crossing Based on Cost Differences.—Mr. Winick attacks the values placed on running and waiting time in that here, too, some optimism may creep in; that the values are not constant; and that the resulting variations are too great to be useful.

In the first place, the term "constants" in the formulas does not mean values that are actually constant. That term refers to averages of variables whose variation is not too large. In the paper, the writer stated that monetary values would vary from time to time, from place to place, and under different conditions. The corresponding ratings would be affected in several ways.

Assume, for example, that in connection with one origin and destination study, Mr. Winick's "executive" and "young lover" were interviewed on their respective trips in each direction, that both had started from the same zone of origin and gone to the same zone of destination and had returned. Assume, also, that both went via the George Washington Bridge in the one direction but that on the return trip the "executive" returned via the George Washington Bridge and the "young lover," having only 25 cents left, had to return via the nearby competing ferry. Assume that both paid 25 cents more toll than that required by the nearby ferry and, in traveling over the George Washington Bridge, had saved 20 minutes. On the original trip, therefore, each paid 1.25 cents per minute saved. If there were only those four trips on that "line of travel," the result would show that of these four trips along this "line of travel," three trips, or 75%, were made via the George Washington Bridge at a cost of 1.25 cents per minute saved. The "executive" probably obtained a bargain rate via the bridge, although the "young lover" might also have paid more in order to make his date on time. How much more they would have paid per minute, they themselves would find it difficult to say unless they were actually faced with a situation of having to pay, say, 50 cents extra or 2½ cents per minute. The "young lover" might even then have chosen the bridge to make his date. However, it is unnecessary to know how much saving in time is probably worth. The significant result is to determine what motorists have actually paid for convenience and to save time, etc., on a super-route, on the basis of the extra tolls that they had to pay.

In fact, in 1930, just such figures were prepared by the writer from origin and destination studies in that year to show the percentages of "lines of travel" which the Holland Tunnel was then handling, in competition with all competitive ferries combined as a group at varying costs per minute saved. (The George Washington Bridge was not opened until late in 1931.) This is a very simple way of expressing the results where there are only two crossings or, as in this case, a tunnel and a group of ferries. As soon as a third or fourth crossing becomes involved, this method of arriving at the percentage handled by any given crossing, on the basis of time and cost, must be expanded into the rating formula (Eq. 5) since the percentage is affected by the number as well as the quality of crossings, expressed by the number and values of their ratings.

Furthermore, applying that very simple procedure of relating percentages of "lines of travel" handled by the Holland Tunnel at varying costs per minute saved, it was also discovered that percentages of some "lines of travel" were obtained by the Holland Tunnel in cases where there was no saving whatever in time or distance. Motorists paid 25 cents more than the ferry tolls, presumably on account of the extra convenience of the tunnel rather than by reason of the savings in time or distance. Hence, it appeared necessary to provide a "constant" to reflect this convenience or inconvenience, in connection with any given crossing. These "constants" were termed "preference" or "prejudice" factors. In some cases, the analysis revealed that the "preference" and "prejudice" factors stemmed not from the crossings themselves but from the travel conditions along the highway routes along the "lines of travel."

If motorists were actually stopped and questioned as to their "preferences" or "prejudices" for or against specific crossings or routes, some would and others would not be able to express the precise reasons; but the varying percentages of the total motorists, traveling along any given "line of travel," that prefer a given route is a subtle and infallible way of determining the mass motorist vote (based on extra tolls paid) as to "preferences" and "prejudices" of given routes. It is the writer's opinion (the contrary opinion of Mr. Winick notwithstanding) "that the same reasons under similar circumstances would motivate other motorists to use the new facility." If one cannot say this, one might as well "throw overboard" all research with respect to mass behavior of motorists.

Use of Empirical Formulas.—Mr. Winick remarks that

> "It takes but little observation to conclude that the annual increment of growth is dependent on too many physical changes to remain constant for any considerable period of years. Similar objections exist in any of the other empirical formulas expressing traffic growth."

The writer agrees but desires to refer back to his statement under the heading, "Measuring the New Traffic: Yardsticks of Traffic Expansion," in which he stated:

> "The foregoing empirical projections relate traffic growth to the mere passage of time and thus impelled the individual assumption of continual growth. They were thus manifestly defective. It was obviously more desirable to relate traffic growth to what might be termed 'traffic determinants,' which would permit anticipation of shrinkages as well as expansions in traffic volumes."

At the same time, it must also be noted that the traffic analyst must still rely, in many cases, upon empirical projections because he has not done sufficient research in any given locality to have developed better methods.

Relative Patronage Factors.—Mr. Winick states that "Closer analysis of relative patronage factors would indicate that they are not only psychological and economic in nature but geographical and social as well." This is quite true. It should be emphasized, however, that the monetary value of the factors, at the given time and in the given locality, will be determined from the studies of origin and destination of traffic in relation to the route characteristics. Such studies would show, for example, that time was worth more in New York than in Iowa. The writer never intended to leave the impression that the monetary values of time, distance, and other factors determined in New York should be applied in Iowa.

Trip Cost.—It is not the total or part of the cost of vehicle operation, but the monetary values that motorists place on route characteristics that determine their choice of routes. Trip costs, as used by the writer, represent not the usual cost of vehicle operation in connection with the trip (although expressed in cents per mile or minute), but the values placed by the motorist on the "resistances" of given routes over the standard, as determined from correlations between usage of routes, expressed in traffic volumes along "lines of travel," and route characteristics evaluated in dollars and cents. Even then the monetary values of time, distance, etc., are only used for weighting the effect of each of these factors to arrive at a composite single "resistance" factor, expressed in cents. Ratings of individual routes then vary inversely as the monetary values of route "resistances." The significant facts about these monetary evaluations are not the absolute values placed by the writer on the factors of distance, say, at 3 cents per mile, running time at one cent per minute, waiting time at 2 cents per minute, and preference and prejudice factors running from 5 to 15 cents, but the fact that, apparently, motorists, in the aggregate, consider a resistance of 2 extra miles of travel equivalent to a resistance of 6 extra minutes of running time, or 3 extra minutes of waiting time.

The writer has also observed that extra mileage as a "resistance" factor is apparently being given less and less weighting, while extra time, especially extra waiting time, is being given more and more weighting as a "resistance" factor.

"Generated" Traffic.—Mr. Winick uses the term "generated" in a different sense from that used by the writer, who defines "generated" traffic as that extra traffic which a new crossing develops in its first year of operation that would not have manifested itself on existing alternate routes if the new one were not opened. Any further expansion in traffic in the second and subsequent years is considered by the writer as part of the regular trend in traffic.

Traffic Determinants.—Mr. Winick states that

> "The determination of future traffic by such determinants as gasoline consumption, department store sales, payrolls, purchasing power, or what have you, is as good a method as is a forecast based on past performance; but even such determinants are subject to wide and unforeseen variation."

Such traffic determinants serve only to guide the traffic analyst in his visualization of the rate of expansion of the entire reservoir of traffic rather than anticipated expansion in the traffic of any one facility whether new or old. It will be noted that, in Fig. 8, the writer used these determinations not in connection with traffic in the Holland Tunnel, or the George Washington Bridge, or the Lincoln Tunnel, but in connection with trans-Hudson traffic as a whole that included the traffic of not only these three trans-Hudson crossings but the traffic of all competing ferries, as well. Trans-Hudson traffic as a whole constitutes the traffic pool from which each of the individual Hudson River crossings will draw, depending upon their individual route characteristics along the "lines of travel" which they serve.

Mr. Winick further states: "In short, it would seem unsafe to forecast traffic for a newly proposed facility on such determinants." The writer again calls attention to his statement (heading, "Measuring the New Traffic: Economic Conditions"):

> "At the time vehicular traffic volumes are projected into the future, it is nearly impossible to foresee the probable changes in local economic conditions. These relationships, however, have served to explain the rate of traffic expansion in the past, and suggest that greater allowances in traffic trends will have to be made in the future for the effects of business conditions."

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TRANSACTIONS

Paper No. 2110

SEALING THE LAGOON LINING AT TREASURE ISLAND WITH SALT

BY CHARLES H. LEE,[1] M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. G. B. BODMAN, JOHN D. WATSON, JOHN W. PRITCHETT, AND CHARLES H. LEE.

SYNOPSIS

Fine-textured earth materials, such as clay or clayey sand and gravel, are potentially impervious to water. Sometimes, however, experience in the use of such material for watertight construction has been disappointing. In some cases, clay membranes placed with the greatest of care have been found to be semi-pervious and have failed to perform the function for which they were designed. This paper describes such an experience at Treasure Island, the site of the Golden Gate International Exposition in San Francisco Bay, California, and the simple method in which the defect was remedied by utilizing an electro-chemical phenomenon of colloidal clay.

The paper describes the novel method by which the 10-in. clay lining of the bottom of the seven-acre fresh-water Lagoon was sealed by a priming of salt water pumped in from the bay. The lining was compacted by use of a 14-ton flat roller, but with comparatively low average density and inclusion of considerable air. Initial seepage loss from fresh water in a test pool was 1.00 in. per day. This was reduced to 0.10 in. per day by the salt water treatment described herein.

DEFINITIONS

Adsorption.—When a gas or solution is brought into contact with a very finely divided or porous material, the pressure of the gas or the concentration of the solution decreases, the gas or solute becoming attached to the surface of the solid. This is the electro-chemical phenomenon known as "adsorption." The gas or solute can be removed from the surface of the solid by exhaustion, heating, or washing. The efficiency of an adsorbent depends largely upon its specific surface (area of surface per unit mass). The forces of adsorption correspond closely with chemical forces or affinity.

NOTE.—Published in February, 1940, *Proceedings*.

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Adsorption Compounds.—The union of atomic or ionic aggregates or mobs in indefinite proportions, under the influence of the forces of adsorption, produces "adsorption compounds." They result from the uniting of particles which have not attained proper subdivision and proximity through solution, fusion, ionization, or pressure to permit the combination of isolated atoms or ions in true chemical proportions. In soil they occur as colloidal soil particles acting as anions (negatively charged) and holding upon their surfaces, by adsorption, ions acting as cations (positively charged) which have been drawn from the surrounding soil solution.

Base Exchange.—A chemical reaction in which two bases or metals exchange places (one being ionized in solution and the other chemically combined as a molecular constituent of an insoluble solid) is known as "base exchange." The process is reversible, the direction depending upon the molecular concentration of the solution. The phenomenon is illustrated by the action of natural zeolites such as sodium silicate in removing calcium and magnesium from hard water which passes over them and giving up sodium to the solution. In soils containing adsorption compounds it may be illustrated by the replacement of calcium ions attached to colloidal soil particles, by sodium ions from the surrounding soil solution.

Calcium Clay and Sodium Clay.—In "calcium clay," calcium predominates, and in "sodium clay," sodium predominates, among the cations resulting from the dissociation of dissolved salts in the soil-water solution. The finer clay and colloidal particles in such a material, acting as negatively charged anions, hold upon their surface the calcium (or sodium) and other positively charged cations adsorbed from the soil solution.

A calcium clay has low sticky properties and the colloidal particles are well flocculated or grouped into aggregates as flocs or crumbs which act somewhat like grains of silt or fine sand and impart a semi-pervious character to the mass. In agricultural soils and fresh-water sediments, calcium is normally the predominant exchangeable ion associated with the clay.

A sodium clay, on the other hand, is very sticky, and with a soil solution of low concentration its colloidal particles are well deflocculated or dispersed into the ultimate minimum size, rendering it impervious to the transmission of water. It is the characteristic clay in salt and alkaline soil and in sediments formed under marine, estuarine, and deltaic conditions.

METHODS OF RENDERING CLAY IMPERVIOUS

Impervious clay beds are of frequent occurrence in nature. The most common physical processes by which such condition is attained are: (*a*) By compaction to a high state of density under weight of superimposed material or long-continued pressure; and (*b*) by complete dispersion of clay and colloids through washing with sea water or impregnation with alkali salts.

Utilizing the first of these natural processes, man has rendered clay impervious, from time immemorial, by the process of puddling, which consists of working clay, or a graded mixture of clay, sand, and gravel, with water to render it compact and impervious. The material receives a preliminary grinding in a pug mill or otherwise the solid particles are thoroughly mixed with water, followed by tamping, ramming, or kneading, as with the trampling of animals, so as to produce high density in the mass. Puddling brings the particles into closer contact, by expelling air and interlocking the granular particles, to the point where the forces of molecular attraction are brought into play. Colloidal materials, including the colloidal aggregates of calcium clays, are rearranged and compacted, thus filling the inter-granular voids and rendering the material permanently impervious to water. With modern construction methods the tamping and kneading action required in puddling is obtained mechanically by use of the sheepsfoot roller.

The second natural process by which clays are made impervious has been recognized only in recent years, and its practical application in construction is still in the experimental stage. The process consists of treating the clay with a sodium salt such as NaCl, or Na_2CO_3, to render it more cohesive and impervious to water. This is accomplished through the chemical process of base exchange by which the calcium ions attached to colloidal particles are replaced by sodium ions from the soil solution. After leaching with fresh water to remove excess sodium ions from the soil-water solution, the colloidal aggregates disperse, changing the physical properties of the soil and filling the voids with a sticky colloidal gel which is impervious to water.

This treatment is applicable, with most success, principally to soils and sediments containing calcium clays in which the "silica sesquioxide ratio" or ratio of silica to iron plus aluminum oxide is high (2 to 7). Such soils are predominant in cool, humid localities such as those in the northern and central regions of the United States and along the Pacific Coast. In the warm humid southeastern states and in the humid tropics, soils with low ratio (2 or less) are common, and sodium treatment has little effect upon physical properties. Residual soils derived from certain igneous and volcanic rocks also show little or no change with sodium treatment.

Sodium treatment is the reverse of the process commonly used in agriculture to improve the working qualities of a heavy sticky soil. For the latter purpose calcium is added to the soil in the form of lime ($CaCO_3$), or gypsum ($CaSO_4$). The purpose is to replace the sodium with calcium and thus soften the soil and render it more pervious by giving it a crumby structure. A familiar example of the application of this process on a large scale occurs in the land reclamation which has been in process for many centuries along the north coast of Holland. The silt deposited by the sea is allowed to accumulate until there is a sufficient area to justify the building of a dike to prevent further incursions of the sea. The finer-textured material which is deposited, having been in contact with sea water, is essentially a sodium clay, but in addition there is a large percentage of calcium carbonate particles. After a lapse of a year or two, surplus sea water is mostly removed in the drainage water derived from rainfall and river overflow. The calcium carbonate then reacts with the sodium clay, gradually converting it into a calcium clay which is capable of successful agricultural production.

Although calcium treatment is common in agricultural practice, sodium treatment, its reverse, has had only experimental application in engineering and construction practice. The first instance, which attained widespread notice, was the use of soda ash in rendering watertight the cutoff wall in the Alexander Dam on the Island of Kauai, Hawaii.[2] This application was entirely successful.[3] More recently salt has been used in road construction as an admixture to stabilize and render impervious earth road surfaces.[4] This process is still in the experimental stage but results appear favorable. The use of sea-water to seal the clay lining of the Lagoon at Treasure Island was probably the first successful use of salt in hydraulic construction. The work was done under the writer's direction, drawing upon a background of investigation and experimental work beginning in 1930.

TREASURE ISLAND LAGOON

An important feature of the landscaping at Golden Gate International Exposition on Treasure Island was the Lagoon, an artificial fresh-water lake with indented shore lines divided into three separate basins interconnected by narrow channels spanned by arched bridges (see Fig. 1). Water lillies and other aquatic vegetation grew in submerged boxes near the edges, and the adjacent margins were sloped and planted with appropriate shrubs and flowering plants.

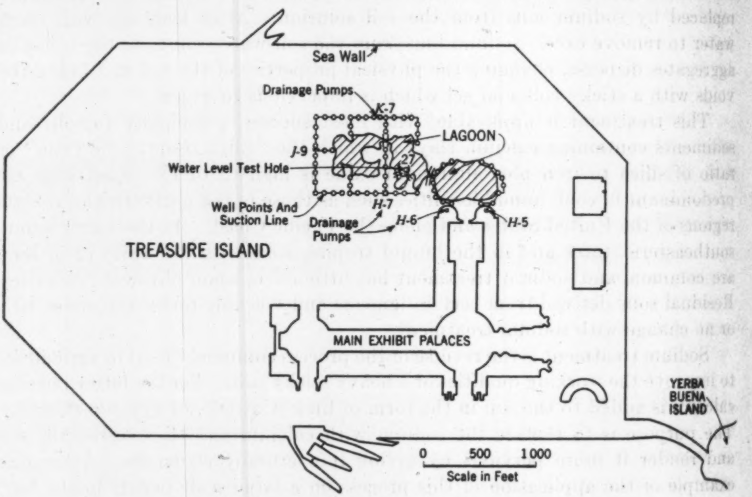
Sea Wall
Drainage Pumps
K-7
J-9
LAGOON
27
Water Level Test Hole
Well Points And Suction Line
H-7
Drainage Pumps
H-6
H-5
TREASURE ISLAND
MAIN EXHIBIT PALACES
YERBA BUENA ISLAND
0
500
1000
Scale in Feet

FIG. 1.—LOCATION MAP, TREASURE ISLAND LAGOON

IMPERMEABLE LINING REQUIRED

Horticultural development required that fresh water, drawn from the main water system that served the Exposition grounds, be used to fill the Lagoon. The Bay Bridge pumping plant supplied it at the average rate of 1.8 mgd from the San Francisco municipal system, through a 10-in. pipe line laid over the suspension spans of the San Francisco-Oakland Bay Bridge to a 3,000,000-gal reservoir on Yerba Buena Island. From that island a 20-in. pipe line across the Causeway fed the distribution network on Treasure Island.

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[2] "Construction Control on Hawaiian Hydraulic Fill Dam Based on Physical Chemistry," by Joel B. Cox, Assoc. M. Am. Soc. C. E., *Hydraulic Engineering*, December, 1929.

[3] "Collapsed Alexander Dam a Notable Structure," *Engineering News-Record*, April 24, 1930, p. 703.

[4] "Salt-Stabilized Road Practice Developing Rapidly," *Engineering News-Record*, July 4, 1935, p. 11.

The capacity of the Lagoon, when filled to its maximum depth of 3 ft, was 7,300,000 gal, and its superficial area was 7.62 acres. The daily evaporation rate from an open water surface on the shores of San Francisco Bay exceeds ¼ in. during the summer months. This represents a loss of 50,000 gal per day from the surface of the Lagoon.

The dredger-fill material that composes Treasure Island is largely medium to fine sand, with an occasional admixture of clay. It is quite permeable and absorbs surface water at a rate exceeding 6 in. per day. The combined daily loss by evaporation and seepage from the Lagoon, if unlined, would exceed 1,250,000 gal. As the water supply facilities were limited by the size of the bridge pipe line, it was necessary to provide an impervious lining for the Lagoon in order to include it as a landscaping feature of the Exposition.

CONSTRUCTION PLANS

Various materials for sealing the Lagoon bottom were considered, including concrete, "bitumels," bentonite, and clay. The elements of effectiveness and economy were considered, and clay was chosen as meeting both requirements. The plans provided for excavation of three basins to a depth of 5.5 ft below the general surface, with side slopes 1 on 3. The elevation of the bottom of the excavation was + 7.5 ft above mean lower low water in San Francisco Bay. The clay seal was to have a finished thickness of 10 in. on the bottom, and for 6 ft up the side slopes a depth of 1 ft. Above El. + 10.0 ft the slopes were covered with a 2-in. minimum layer of gunite with wire mesh extending up to El. + 12.5 ft (see Figs. 2 and 3). Rock riprap was to be laid just above and below flow line at El. + 11.5 ft. The estimated quantity of material in the clay lining was 11,020 cu yd compacted in place. There are 62,000 sq ft of gunite protection and 18,500 sq ft of riprap.

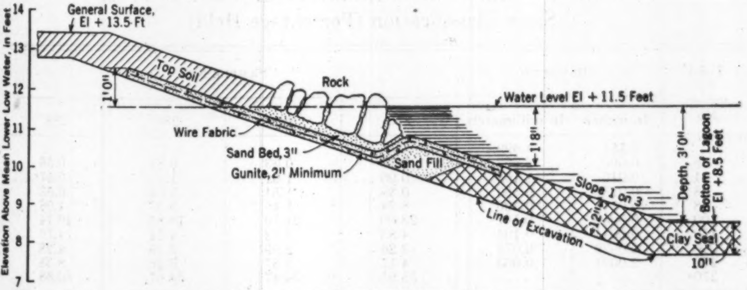

FIG. 2.—TYPICAL SECTION OF LAGOON LINING

SELECTION OF CLAY

The specifications as prepared by the Construction Division of the Exposition Company's Department of Works required that clay for the lining should be "a natural mixture of clay and sand occurring naturally in the borrow pit offered by the contractor," and that the character of the clay offered would be a governing factor in the award of the contract. The contractor was required to submit samples of material proposed for use. It was specified that the lining material should contain not less than 35% nor more than 65% of clay, the remainder to be sand, and that it should be satisfactory for producing a watertight layer after consolidation.

FIG. 3.—SIDE SLOPE OF LAGOON UNDER CONSTRUCTION

TABLE 1.—MECHANICAL ANALYSES OF CLAY SAMPLES
Sieve Classification (Percentage Held)

Held by screen No.	DIAMETER		SAMPLE NO.:			
	In inches	In millimeters	279	280	281	288
....	0.181	4.699	....	....	....	....
8	0.093	2.362	0.10	0.33	0.88	0.58
14	0.046	1.168	0.06	1.01	1.21	0.57
28	0.0232	0.589	0.39	1.54	1.64	0.53
48	0.0116	0.295	8.24	8.46	8.37	4.08
100	0.0058	0.147	23.00	21.10	18.85	16.14
150	0.0041	0.104	4.83	3.73	4.84	4.76
200	0.0029	0.074	3.26	2.99	3.28	4.28
270	0.0021	0.053	4.17	6.37	6.26	8.38
270[a]	....	....	55.95	54.47	54.67	60.68
Total	....	....	100.00	100.00	100.00	100.00

[a] Passing Sieve No. 270.

In compliance with the specifications, samples of clay submitted from 4 borrow pits in various parts of the San Francisco Peninsula were tested in accord with the specifications. The mechanical analyses data (see Tables 1 and 2) indicated that Sample 288 conformed most closely to the size specification. Graphical plotting showed also that the material in this sample approached more nearly to the ideal grading of sizes[5] for maximum density and impermeability than any of the other samples.

TABLE 2.—SIZE CLASSIFICATION (PERCENTAGE BY WEIGHT)

Classification	Diameter, in millimeters	SAMPLE NO.: 279	280	281	288
Gravel	2+	0	0.5	0.6	0
Fine gravel	2–1	0	1.2	2.2	1.0
Coarse sand	1–0.5	2.2	2.9	4.2	1.3
Medium sand	0.5–0.25	11.8	9.6	12.7	6.7
Fine sand	0.25–0.1	23.0	22.4	16.0	18.2
Very fine sand	0.1–0.05	8.6	10.3	9.5	12.8
Silt	0.05–0.005	34.1	29.1	29.8	27.8
Clay	0.005–	20.3	24.0	25.0	32.2
Total	....	100.0	100.0	100.0	100.0

Compaction tests were run on three of the samples following the procedure outlined by R. R. Proctor,[6] M. Am. Soc. C. E., with the results shown in Table 3.

Although these tests showed that Sample 288 had slightly less density at optimum moisture content than the other samples (see Fig. 4), the difference was small and it was concluded that the greater clay content would insure a greater degree of impermeability for Sample 288 than for the others. This sample was recommended, therefore, but it was emphasized that use of a sheepsfoot roller would be necessary in compacting it in order to secure an impermeable membrane. Arrangements were finally made with the contractor to use material from the borrow pit from which Sample 288 was obtained.

TABLE 3.—SUMMARY OF COMPACTION TESTS (AT OPTIMUM MOISTURE CONTENT)

Description	SAMPLE NO.: 279	281	288
Dry weight, in pounds per cubic foot	122.5	120.0	118.0
Moisture, expressed as a percentage of dry weight	12.4	14.0	14.5
Plasticity needle reading, in pounds per square foot	1,300.0	680.0	990.0
Air voids, expressed as equivalent moisture percentages	1.7	1.2	1.4
Porosity (percentage by volume)	27.8	29.6	29.8
Allowable working moisture range [a] (percentages):			
From	12.4	14.0	14.5
To	14.9	15.3	17.0

[a] Based on a minimum plasticity needle reading of 300 lb per sq in. required to support heavy construction equipment.

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[5] "Selection of Materials for Rolled-Fill Earth Dams," by Charles H. Lee, *Transactions*, Am. Soc. C. E., Vol. 103 (1938), Fig. 23, p. 56.

[6] "The Design and Construction of Rolled Earth Dams," by R. R. Proctor, *Engineering News-Record*, August, 31–September 7, 1933, pp. 21, 28.

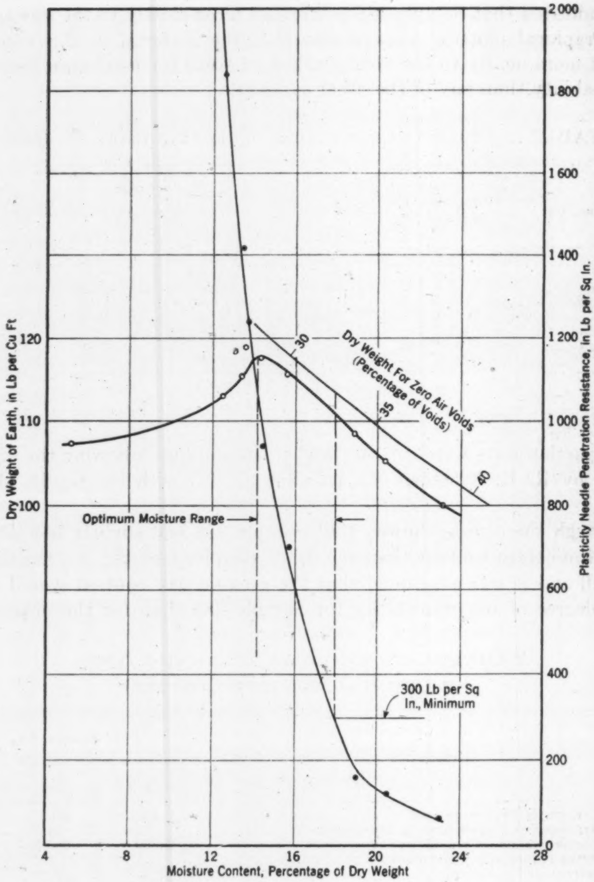

FIG. 4.—COMPACTION DIAGRAM—SAMPLE 288

TABLE 4.—CHEMICAL ANALYSES FOR EXCHANGEABLE BASES OF CLAY SAMPLE 288

Description	EXCHANGEABLE BASES					
	Calcium	Magnesium	Sodium	Potassium	Hydrogen[b]	Total capacity[c]
Exchangeable base capacity[a]........	6.8	3.6	0.6	Trace	0.2	11.2
Percentage of total capacity........	60.7	32.1	5.4	....	1.8	100.0

[a] Milligram equivalent per 100 g of dry sample. [b] By difference. [c] Total capacity for exchangeable bases.

The material is described as a brownish yellow clay. It is decomposed Cahil sandstone lying in place as a 10-ft layer upon the bedrock that forms a low flat-topped hill in the City of South San Francisco approximately one half mile from San Francisco Bay. The clay was in a more or less flocculated condition and could be handled without difficulty. The pH-value was practically neutral, tests running from 6.9 to 7.2. A chemical analysis of the soil for exchangeable bases, as made by the Division of Soil Technology, University of California, Berkeley, Calif., showed it to be a calcium clay, typical of fresh-water conditions (see Table 4).

Lining Operations

The material was excavated by power shovel after stripping the top soil, care being exercised not to go below the level of complete decomposition of the bedrock. Transportation was by barge to Treasure Island and truck to the Lagoon site. The material was spread to uniform thickness in two successive layers, each one being thoroughly rolled with a 14-ton flat road-roller. The latter was required to make a minimum of four passes and to continue until the mass was consolidated satisfactorily and until no movement of the clay appeared in front of the roller. The thickness of the first layer after compaction was 6 in. to 7 in., and the total thickness of both layers, 10 in. or more.

Material was sprinkled with water as dumped from the truck, but no effort was made to attain a condition of uniform moisture before rolling. Those in charge of construction considered that the sand subgrade had insufficient stability to support the first layer of clay under the concentrated weight of the roller, and that the feet of a sheepsfoot roller would push through into the sand and break the continuity of the clay membrane. Therefore, this type of equipment was not used.

Water was applied daily by hand sprinkling upon the finished rolled surface, both first and second layers. The first layer was exposed to the air for only a few days before spreading the second layer, but the latter was exposed nearly two months—from the middle of August until the first filling of the Lagoon on October 18. Where the bottom layer had stood for several days, systematic shrinkage cracking was evident, cracks having a depth of 2 in. or more. During the long period of exposure prior to flooding, there was more extensive cracking in the top layer. Cracks formed with typical geometric patterns, principally five-sided polygons, and in some cases attained considerable width and depth although apparently limited to the top layer.

Tests of Completed Clay Lining

Tests made under the writer's direction upon samples of the clay material immediately after rolling showed moisture content varying from 15% to 19%, and in a few cases 25%. After the lapse of one or two days, tests of rolled material showed moisture content of 13% to 16% with a hard crust.

Density tests were made also. Samples were taken from rolled material by driving, vertically, a 3-in. metal cylinder 15 in. long with sharp cutting edge to a depth of 6 in. or more, the length of the sample in place being measured by establishing a reference point outside the sampling tube and measuring from the reference point to the top of the tube before and after driving. In this manner, error due to compaction in driving was eliminated. Volume of sample was computed from internal diameter of tube and length of sample. The complete sample was weighed before and after oven drying.

TABLE 5.—DENSITY TESTS

Sample No.	Location	Moisture content (percentage by weight)	Density; dry pounds per cubic foot
290	Near southwest corner of Lagoon 3; first layer several days after rolling	13.3	73.4
291	Near center Lagoon 3; first layer two days after rolling	15.4	69.0
297	Near northeast corner Lagoon 3; top layer at time of rolling	16.55	104.4
298	Near center Lagoon 3; top layer one day after rolling	13.74	88.7

The results of density tests shown in Table 5 indicate low density with inclusion of much air in the clay lining. The latter was due largely to the use of a flatwheel roller, which compacted a surface layer immediately beneath the wheel but did not consolidate the lower part of the layer adequately. This procedure sealed the surface and trapped the air in the lower portion of the layer, where it was held by the cohesion of the clay, probably with pressure greater than atmospheric.

Four tests made by others, two weeks after completion of the top layer, showed moisture contents of 9.9 to 11.7%, and dry weight densities of 125.7 to 133.1 lb. These samples were taken from the center of the top 4 in. and bottom 6 in. after drying and shrinkage had occurred. A 2-in. by 2-in. cylinder was used and the length of sample was obtained by measuring the final length of core cut by the tube. The dry weights as thus determined were in excess of the true values at the date of sampling due to the compacting of the sample in driving. The true values, if available, probably would have exceeded those obtained by the writer about two weeks earlier and prior to compaction by shrinkage.

Test of seepage loss through the clay lining was made September 6 to 12, 1938, about three weeks after completion, by levying off an area 45 by 60 ft and filling it to a depth of 13 in. with fresh water. The daily rate of drop in water level was uniform throughout the period of test, amounting to 1.04 in. Average daily evaporation during this period did not exceed 0.14 in., the latter being recorded at an insulated evaporating pan kept by the California Forest and Range Experiment Station, U. S. Forest Service, at the mouth of Strawberry Canyon, Berkeley, about 5 miles east of Treasure Island. Evaporation by the record is slightly in excess of that from a large water surface at Treasure Island, so that net loss due to seepage was not less than 0.90 in. per day or 187,000 gal per day over the entire Lagoon area. This was much in excess of the allowable seepage rate and it was apparent that further steps would have to be taken in order to obtain a watertight lining.

In addition to a high leakage rate, the clay was found to have softened to a mush for a depth of 8 in. or more, due to the penetration of water into the semi-pervious clay. The internal cohesion was thus reduced to the point where trapped air could expand and escape, thus breaking and opening up the soil structure. This condition was also very unsatisfactory as it would permit the easy penetration of poles or any hard object coming in contact with the bottom.

TESTS OF SALT TREATMENT

Immediately after it was demonstrated that the membrane as rolled was not satisfactory, the writer made tests to determine the effectiveness of salt treatment in hardening the clay and rendering it impervious in the presence of fresh water. Four test samples were prepared from dry material by crushing and mixing with fresh water to a stiff plastic consistency which would hold its shape without slump, and then rolled between the hands into balls 2 in. in diameter. To the material in the first ball nothing was added, but to the other three balls salt was added, before mixing, in an amount which when dissolved in the mixing water would give the approximate salt equivalent of half-strength, full-strength, and double-strength sea water.

After drying in the laboratory for three days, the balls were in the following condition: No. 1 and No. 2 were hard and dry with light color, showing moisture film had withdrawn into ball; No. 3 was hard and partly dry with color still fairly dark; and No. 4 was firm, but very slightly plastic with dark color.

The balls were then submerged in fresh water to two thirds of their diameter. After submergence for 2 hr, the following was observed: No. 1 was very soft throughout and two thirds disintegrated with much formation of small air bubbles; No. 2 was slightly softened and about one third disintegrated below water surface with much formation of air bubbles; No. 3 showed no change above the water surface, but slight scaling and a few air bubbles were evident below the water surface, with accumulation on the bottom of the pan; and No. 4 showed no change.

After 12 hr of submergence, No. 1 was completely disintegrated, but no change was observed in No. 3 or No. 4. The same condition was found to continue indefinitely thereafter.

The rapid scaling and disintegration of the ball without salt was caused by the explosive action of entrapped air as it was being displaced by capillary water. The slower disintegration or absence of disintegration in the salt-treated balls was caused by the process of base exchange which changed the clay from a calcium to a sodium type, breaking up the crumby structure with dispersion of particles to ultimate size and formation of a sticky colloidal gel. In this condition, capillary water did not enter the ball when submerged, thus preventing softening and disintegration.

This test, when repeated in the presence of those responsible for expenditures, proved to be very convincing, and decision was made to try salt as a sealing agency.

SEA-WATER PRIMING

Before adopting a procedure for salt treatment, consideration was given to two questions: (1) Whether sea water would be as effective as crude salt; and (2) how much sea water would be required to insure replacement of calcium by sodium in the clay lining.

A sample of sea water from the Pacific Ocean near Cliff House, San Francisco, was taken by the Division of Water Resources,[7] State of California, on August 7, 1929, and chemical analysis made, with the results shown in Table 6. This sample is representative of water in San Francisco Bay surrounding Treasure Island during the fall of the year when flow of fresh-water streams tributary to the Bay is at a minimum. The presence of appreciable calcium and magnesium was recognized as a possible source of difficulty in making rapid replacement of these elements in the soil by use of sea water. The cost of pumping sea water from the Bay was far less than the use of crude salt, and with the knowledge that clay beds washed with sea water are always in a deflocculated and impervious condition, decision was made to use sea water.

TABLE 6.—CHEMICAL[a] ANALYSIS OF SEA WATER FROM PACIFIC OCEAN NEAR CLIFF HOUSE, SAN FRANCISCO, CALIF.

Description	Na, K	Ca	Mg	SO_4	Cl	CO_3	HCO_3	Totals
Milligram equivalents per liter	490	21	99	52	512	0	2.6	1,151.6
Parts per million	11,275	426	1,212	2,500	18,200	0	159.0	33,394.0
Percentage of total	33.3	1.3	3.6	7.4	53.7	0	0.5	99.6

[a] Na, sodium; K, potassium; Ca, calcium; Mg, magnesium; SO_4, sulfate; Cl, chloride; CO_3, carbonate; and HCO_3, bicarbonate.

Very little information was available as precedent for determining the quantity of sea water required. Tests had been made at the University of California to determine the effect upon permeability to water of a Yolo silty clay loam soil from near Davis, Calif., of applications of irrigation water possessing various compositions and degrees of purity. Water containing ions of sodium, calcium, chloride, and sulfate, in the total amount of 4,000 ppm, and in which the ratio (by chemical equivalents) of sodium to calcium was 4 to 1, was used in one such test. Surface application of this water to a 6-in. column of soil, 2 in. in diameter, in an amount corresponding to 20 times the void space of the soil, increased the soil permeability during a percolation period of 17 days from an original value of 1.2 in. per day to 7 in. per day, at which time washing by percolation was begun with salt-free water. Within 2 hr after the removal of the salt solution, the beginning of a change in the entire soil system had decreased the permeability to 1 in. per day. During the next three days the permeability had decreased still further, to 0.06 in. per day, and finally after 49 days of continuous leaching with salt-free water the low permeability of 0.02 in. per day was attained.

The ratios of chemical equivalents of replaceable ions in the soil, and of corresponding ions in the irrigation water and in the clay lining of the Lagoon and sea water are given in Table 7.

The sodium-calcium ratio of the sea water was approximately five times that of the irrigation water. In addition, the sodium-calcium ratio of absorbed bases in the clay lining was approximately twice that in the Yolo silty clay loam and the total absorptive capacity was less in the case of the clay lining. Furthermore, the total sodium concentration in the sea water was many times greater than in the irrigation water.

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[7] "Variation and Control of Salinity in Sacramento-San Joaquin Delta and Upper San Francisco Bay," Calif. Dept. of Public Works, Div. of Water Resources, *Bulletin 27*, 1931, Table 36.

An application to the Yolo soil of the irrigation water concerned corresponding in volume to 20 times the soil void space (for a void ratio of 1) represented

TABLE 7.—COMPARATIVE RATIOS OF CHEMICAL EQUIVALENTS

Material	Sodium	Magnesium	Calcium
Yolo silty clay..............................	1	30	27
Irrigation water..............................	4	0	1
Clay Lagoon lining..............................	1	6	11
Sea water..............................	23	5	1

$\frac{1{,}143 \times 62.5 \times 20 \times 0.5}{1{,}000{,}000}$ lb per cu ft = 0.71 lb of sodium per cu ft of soil, whereas the same application of sea water to the clay lining would represent $\frac{11{,}275 \text{ parts} \times 62.5 \text{ lb} \times 0.4 \times 20}{1{,}000{,}000}$ lb per cu ft = 5.60 lb of sodium per cu ft of clay. The latter application, if accumulated at the surface of the clay lining, would be 20 × 10 in. × 0.4 = 80 in. deep. In order to insure the availability of ample excess sodium for the replacement of calcium in the clay, plus an excess of water for waste by evaporation, leakage other than seepage, etc., it was decided to pump sea water into the Lagoon until a total depth of 40 in. had escaped. If all sodium in this water were absorbed by the clay, it would repre-

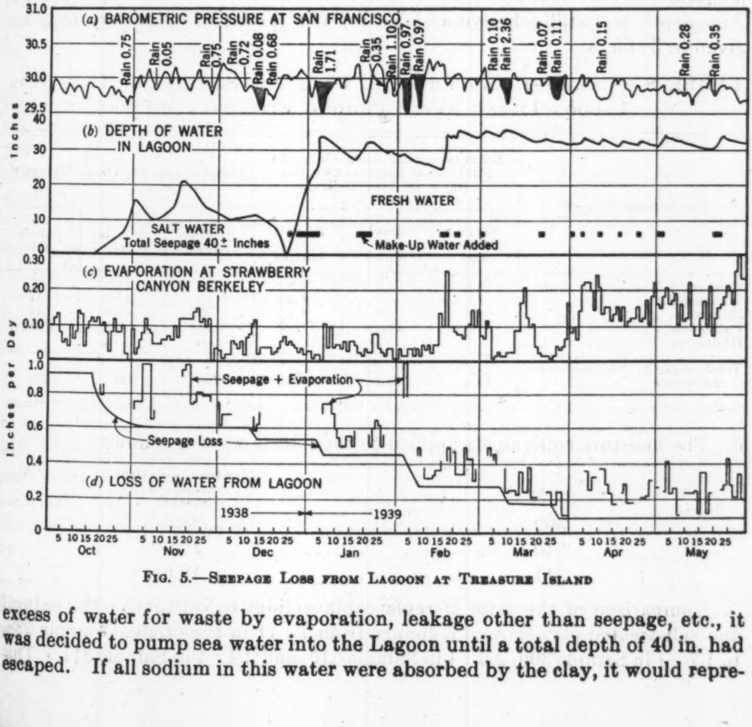

FIG. 5.—SEEPAGE LOSS FROM LAGOON AT TREASURE ISLAND

sent 2.80 lb of sodium per cu ft of clay, or four times that available in the laboratory experiment.

Although this quantity of sea water included a generous margin, it could be provided with little additional cost, since power was being purchased under a general wholesale rate for all Exposition uses and pumping equipment was already on hand.

Sea water from San Francisco Bay was pumped into the finished Lagoon beginning October 20, 1938, and continued until a depth of 18 in. was attained which completely submerged the clay membrane. This was found to produce an immediate reduction in seepage from the rate of 0.90 in. per day with fresh water to 0.60 in. (In Fig. 5, the seepage-loss line is determined by subtracting the observed evaporation at Strawberry Canyon from the loss of water from the Lagoon for periods of four or more days.) Furthermore, the clay, although softening to a depth of 2 in., was hard and cohesive for the remainder of its depth. In order to impregnate the clay lining thoroughly with salt water, pumping was continued until December 24, 1938, when the accumulated depth of seepage had reached a total of 40 in. At this time all remaining salt water was drained from the Lagoon and fresh water run in to a depth of 34 in. (see Fig. 5).

In the interval prior to filling with fresh water, samples were taken for physical and chemical tests, and it was found that the consistency below the 2-in. depth was still cohesive and hard. The results of laboratory tests are given in Table 8.

TABLE 8.—CHEMICAL AND PHYSICAL DATA FOR SAMPLES OF CLAY FROM LAGOON LINING AFTER PRIMING WITH SALT WATER

Exchangeable bases	EXCHANGEABLE BASE CAPACITY MILLIGRAM EQUIVALENT PER 100 g OF DRY SAMPLE			PERCENTAGE OF TOTAL CAPACITY		
	Sample 309 top	Sample 310 center	Sample 311 bottom	Sample 309	Sample 310	Sample 311
Calcium	2.6	2.8	1.8	18	14.5	17
Magnesium	7.6	8.7	4.3	52	45.5	41
Sodium	4.4	7.6	4.4	30	40.0	42
Potassium	Trace	Trace	Trace	Trace	Trace	Trace
Hydrogen	....	....	....	....	....	....
Total capacity for exchangeable bases	14.6	19.1	10.5	100	100	100

The moisture equivalents applied to these tests were as follows:

Sample	Moisture equivalent (percentage of dry weight)
309	23.8
310	26.0
311	19.5

Comparison of the ratio of replaceable sodium to calcium in the natural and salt-treated clay showed a change from 1 : 11 in the original Sample 288, to 1.7 : 1 in Sample 309, 2.6 : 1 in Sample 310, and 2.4 : 1 in Sample 311. This represented a 24-fold increase of sodium over calcium with only a 30% increase in the total replaceable bases. The moisture equivalent tests indicated a material highly retentive of moisture. The tests were thus indicative of extensive replacement of calcium by sodium and gave confidence as to the ultimate reduction of seepage when deflocculation was made possible by the removal of excess sodium in the soil solution by leaching with fresh water.

FRESH-WATER FILLING

Soon after fresh-water filling was begun, it was noted that great quantities of air were held in the upper section of the clay bottom of the Lagoon. This could be released by dragging the end of a rod along the bottom. It came to the surface principally in clouds of small bubbles, but sometimes as large bubbles. No air was noted escaping except when the bottom was disturbed.

The seepage rate during the first month after filling was constant at 0.46 in. per day (see Fig. 5). Suddenly early in the second month, without apparent reason, the rate dropped to 0.28 in. per day and continued thus to the early part of the third month, when a further drop to 0.18 in. per day occurred. Later in the month this reduced to 0.10 in. per day. Upon investigation of the continuous record of barometric pressure at the San Francisco station of the U. S. Weather Bureau, it was discovered that unusually low barometric pressures were recorded just preceding and during each of these periods of decreasing rate (see Fig. 5). It was also noted that after the second low barometric period, the quantity of air held in the top layer of clay was noticeably less

The conclusion is believed to be justified that low barometric pressure in the surrounding atmosphere produced an unbalanced pressure in the clay voids containing trapped air, which was great enough to overcome cohesion and permit release of the air. Such release would suddenly rearrange the particle structure and permit fresh water to enter and wash the walls of voids previously filled with air. This would allow more complete dispersion of colloidal particles and the filling of the voids with sticky colloidal gel. The clay lining thus would be rendered more impervious to water.

The final seepage rate of 0.10 in. per day is within the allowable limit and permitted replenishment of losses from the Lagoon without excessive draft on the Exposition water supply.

GROUND-WATER BEHAVIOR

In connection with seepage, a study was made of the behavior of the plane of saturation in the fill beneath the Lagoon. For this purpose water levels were observed at intervals of a few days in a 3-in. auger test hole lined with perforated casing and located on a sand island which was built on top of the completed clay lining (see Fig. 1). This test hole (No. 27, Fig. 1) penetrated the clay layer and extended beneath it a distance of 10 ft with a total depth of 15 ft. The record of water level in test hole No. 27 is reproduced graphically in Fig. 6, together with relative elevations of clay lining and water surface in the Lagoon. This record shows clearly the combined effects of increased depths of water in the Lagoon and of decreasing permeability of the clay lining.

Continuous drainage of Treasure Island was necessary to prevent rise of ground water to the level of ornamental tree roots. Seepage from the Lagoon was anticipated as one of the important sources of ground water, and in order to prevent excessive local accumulation five drainage pumping units were installed near the Lagoon with well points completely encircling it at intervals of 100 ft (see Fig. 1). By this means the ground-water increment from the Lagoon is drained away, and the water level in the fill surrounding the Lagoon is maintained between El. − 1.0 ft and El. − 2.0 ft.

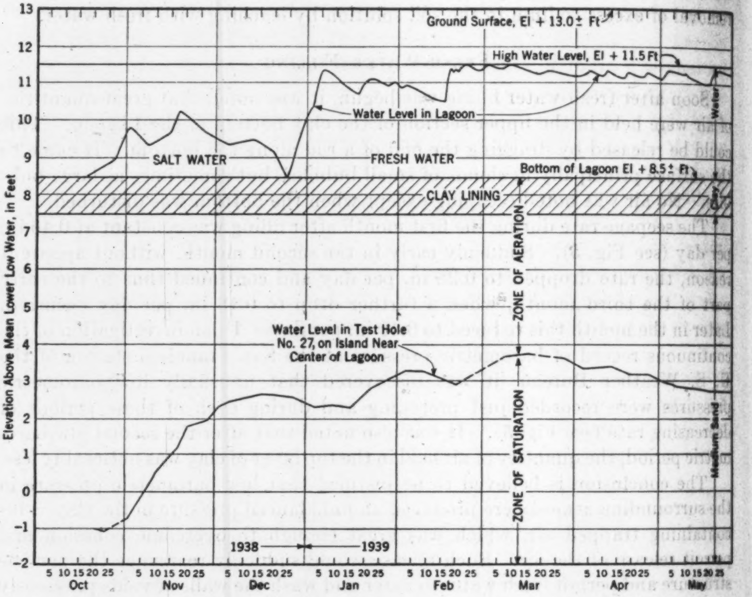

Fig. 6.—Relative Elevations of Water Level in Lagoon and in Underlying Sand Fill

The close relation between seepage and pumping is illustrated by comparing the increased rates of pumping and seepage after the Lagoon was filled. During the eleven weeks prior to filling with salt water (October 18, 1938) the total pumpage from all five drainage units was 4.44 acre-ft per week. The approximate average rate of pumping during the eleven weeks after filling was 7.21 acre-ft per week, an increase of 2.77 acre-ft per week. The rate of seepage during this period (see Fig. 5), if applied to this entire Lagoon area, was equivalent to 2.45 acre-ft per week. As pump records were probably in excess of actual quantities pumped, these values can be considered as in close agreement.

The rise in water level in test hole No. 27 represents the accumulating ground storage under the Lagoon required to produce sufficient gradient for water to flow to the well points. During the first three weeks after filling, the rate of rise was 1 ft per week, which, if it were uniform over the entire area, would represent 1.8 acre-ft per week, using the drainage factor of 0.23 as determined by large-scale field test. Subsequent rate of rise up to the middle of March represented 0.2 acre-ft per week. After that date the water level fell, indicating that seepage was becoming insufficient, even with full Lagoon, to maintain the initially created gradient of the water table.

EFFECT OF SALT TREATMENT UPON QUALITY OF WATER

Tests made upon samples of water, taken from the Lagoon 4.5 months after filling with fresh water, showed no appreciable increase in mineral content over that of the source of supply. The pH-value was practically neutral, ranging from 6.9 to 7.2. Aquatic vegetation grew normally, with roots submerged in the water. Apparently all excess mineral salts, remaining in the soil solution of the clay lining after drawing off of the salt water, were leached out by the fresh water and were carried on down into the underlying sand. As soon as the soil solution was freed of unattached sodium ions by leaching, the clay particles were dispersed, forming a sticky gel that filled all voids. In this form the sodium remains fixed as long as it is in contact with fresh water. The quality of fresh water in the Lagoon, therefore, was unaffected by the salt treatment.

CONCLUSIONS

The clay lining of Treasure Island Lagoon was sealed successfully by salt-water treatment at a cost of a few hundred dollars, without interfering with other necessary operations. To have sealed it by other methods, such as application of bentonite, would have cost at least $15,000 and would have involved delay in completing the landscaping features.

In view of the fact that fresh water used in filling the Lagoon subsequent to the salt-water treatment did not experience an increase in salinity, this method of sealing would appear to have a wide field of usefulness in constructing impervious membranes for water works structures such as reservoirs and dams, as well as those for recreational and other purposes not involving potability of water.

ACKNOWLEDGMENTS

Treasure Island Lagoon was designed and built by the Department of Works, Golden Gate International Exposition, of which W. P. Day, M. Am. Soc. C. E., was director. The Division of Soil Technology, College of Agriculture, University of California, made chemical tests of the clay used in the lining. G. B. Bodman, of this Division, confirmed the writer's conclusions as to efficacy of salt-water treatment, and advised regarding the quantity and methods of application.

Evaporation data were furnished by E. I. Kotok, director, California Forest and Range Experiment Station, U. S. Forest Service, and continuous barometric pressure records and rainfall data were furnished by Edward H. Bowie, meteorologist, U. S. Weather Bureau. Soil testing for the selection of material for the clay lining and experimental work leading to the adoption of salt-water treatment were conducted at Pacific Hydrologic Laboratory under the writer's direction.

DISCUSSION

G. B. **Bodman**,[8] Esq. (by letter).—In the early part of his paper Mr. Lee mentioned two methods of rendering clay beds impervious. The first of these is chiefly mechanical and depends upon so-called puddling processes which produce partial dispersion and compaction. The second method is chiefly chemical and depends upon the application of the physicochemical principles of colloidal chemistry.

Those who are interested in soils because of their agricultural importance are naturally concerned with the fundamental properties of their constituents, not only because of their immediate rôle in plant nutrition, but also because of the somewhat less direct effects they may exert upon plant growth and crop production by way of the physical properties which they impart to the soil mass.

Interest in the physical properties of soils is greatly stimulated, in the west, by a phenomenon which is almost exactly the opposite of that which demands the attention of the engineer. Whereas clay beds, or artificially distributed layers of clay soils, may tend to possess too low a density, too high a porosity, and too great a permeability for the purpose of the engineer, the experience under irrigation agriculture in Western United States is that certain soils may change under irrigation, and tend to become too dense and to assume too low a porosity when cultivated. Furthermore their permeability to, and ease of penetration by, irrigation water may be so greatly reduced that their management is much complicated and their productivity likely to be seriously impaired. The agriculturist must frequently seek, therefore, to overcome the very conditions which the engineer may desire to create.

For these reasons Mr. Lee's experience with the Exposition Lagoon is of special interest to the soil scientist since a condition has been produced which is beneficial from the engineer's viewpoint but which from the point of view of the farmer is harmful and similar to that which may be produced in soils, either in field or laboratory, by the continued use of saline irrigation water high in sodium.

Knowledge of the constitution of mineral clays, such as those now known to occur in soils, is being extended almost daily by workers in various parts of the world. Accordingly the diagrammatic models which have been designed for aid in the interpretation and prediction of the behavior of clays require frequent revision. Certain relatively conservative and simple pictures of the mechanisms may still be utilized, nevertheless, and actually prove most useful in visualizing the behavior of clays under given conditions. It should be mentioned, however, that much of the existing physicochemical information depends upon experience with suspensions of clay. Much less really fundamental work has been performed upon pasty masses of clay in which the ratio of solids to liquids is relatively high.

The clay particles in soils may be considered as particles of colloid each one of which acts like a weakly dissociated molecule of polybasic aluminosilicic acid possessing a high molecular weight. In addition to releasing more or less hydrogen ions when suspended in, or bathed by, water, they also release, in varying proportions, cations of the metals commonly found in soils—namely, calcium, magnesium, sodium, and potassium. These may be replaced by electrically equivalent amounts of other ions and are said, therefore, to be exchangeable. The kind and degree of dissociation are such that the resultant system, while electrically neutral as a whole, may be visualized as possessing a bulky, negatively charged core (the anion) surrounded by a swarm of much smaller, positively charged bodies (the cations). The cations are in a state of continual movement about the anion and are, on the average, more numerous near its surface than they are farther away.

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The magnitude of the negative charge on the particle and also the mean distance between the particle and the concentric shell of cations appear to be measures of the permanence with which a water suspension of clay particles can resist flocculation. The magnitude of the negative charge depends upon the chemical constitution of the particle. The amount of dissociation and the mean distance between negative charges on the particle and the center of gravity of the positive charges which surround it depend upon (1) the kinds of cations present—sodium, if present, being most highly dissociated from the particle, hydrogen being least dissociated, and potassium, magnesium, and calcium, in decreasing order, being intermediate in dissociation; and (2) the abundance and kind of electrolytes, or salts, present in the surrounding water.

The same factors that control the stability of a clay suspension (that is, its resistance to flocculation and to the tendency for its particles to cohere upon impact into clumps or aggregates of considerable bulk and to settle out into a loosely packed mass of high void ratio) appear to exert a profound effect upon the behavior of clay particles in a body of soil. When sodium predominates as the exchangeable cation, the clay particles in a mass of soil impart to it physical properties which are conspicuously clayey. The reverse is generally true if calcium is the predominating exchangeable cation. Since an ionic exchange (termed base exchange) may be effected by sheer force of ionic numbers, soil containing a large number of calcium ions in association with its clay particles may have them exchanged for sodium ions by a sufficiently long leaching with water containing a sufficient number of dissociated sodium ions. The leaching serves the twofold purpose of supplying the sodium ions and removing the displaced calcium ions. As long as the soil pores are occupied with salt-rich water, the exchange of calcium for sodium may produce relatively little effect upon the physical behavior of the soil mass. Upon removal of the excess salts by washing, however, the newly formed sodium clay will dominate the behavior of the soil mass; the aggregates will tend to disperse; the streaming water will cause local migration of the liberated individual clay particles, blocking the finer pores; and the entire mass will become sticky, retentive of water, and highly impermeable. A partial reversal of these conditions may be brought about by replacement with calcium although the cycle of alterations is not strictly reversible.

Utilization of the physicochemical method of reducing the permeability of a soil mass requires the consideration of the following points: (1) The nature of the soil, its composition, and existing base status; (2) the removal of excess electrolytes; and (3) if the desired cation is added in solution, the composition of the water used. Tables 9 and 10 will help to illustrate the significance of these considerations.[9]

TABLE 9.—PERMEABILITY OF YOLO SILTY CLAY LOAM AS AFFECTED BY PROPORTION AND CONCENTRATION OF SODIUM AND CALCIUM CHLORIDES IN THE PERCOLATING WATER DURING BOTH THE PRELIMINARY AND RECLAMATION TREATMENTS

COMPOSITION OF WATER DURING PRELIMINARY TREATMENT					PERMEABILITY (WATER-SATURATED), AS INCHES PER DAY PER ONE GRAVITY; DURING TENTH DAY OF:							
Concentration*	Amounts Present per Liter, of:				Preliminary Treatment with Salt Water				Reclamation with Distilled Water			
	Sodium		Calcium									
In parts per million	Grams	Milligram equivalents	Grams	Milligram equivalents	a	b	c	Average	a	b	c	Average
(*a*) HIGH CALCIUM WATER (RATIO OF SODIUM TO CALCIUM, BY EQUIVALENTS, EQUALS 1 TO 8)												
5000	0.229	9.95	1.595	79.6	13.3	12.8	10.0	12.0	1.2	1.3	1.0	1.2
500	0.023	0.995	0.159	7.96	7.6	7.5	9.5	8.2	0.77	0.85	0.95	0.86
(*b*) HIGH SODIUM WATER (RATIO OF SODIUM TO CALCIUM, BY EQUIVALENTS, EQUALS 8 TO 1)												
5000	1.758	76.5	0.192	9.56	5.0	6.0	6.5	5.8	0.03	0.03	0.03	0.03
500	0.176	7.65	0.019	0.956	4.4	1.7	2.9	3.0	0.27	0.21	0.19	0.22

* Concentration is given in parts of sodium chloride plus calcium chloride per million of water. Thus 5,000 ppm indicates, in the case of high calcium water, that 1 liter of such water contains 5 g of the mixed chlorides in the proportions—0.229 g sodium : 1.595 g calcium : 3.176 g chloride. The less concentrated salt waters having only 500 parts of mixed chlorides per million of water contain, respectively, the same materials in the same proportions but in one tenth the amounts.

Referring to Table 9, the soil columns were first saturated with distilled water from below, and the saline water was then added from above for downward percolation. Before reclamation began with distilled water, flow was interrupted for 17 days, the soil remaining in contact with the saline water during that interval, until the distilled water was added from above, for downward flow. Trials were made in a basement laboratory at an average temperature of 26° C.

The Yolo soil has a rather high clay and silt content and originally is very low in exchangeable sodium. Note that ten days of leaching with more or less saline irrigation water results in a set of permeability values which have magnitudes associated with the abundance of sodium in the water. The water containing 5,000 ppm of calcium and sodium chlorides in the ratio of eight equivalents of calcium to one of sodium tends to maintain a high permeability. The calcium status of the soil is increased and there is relatively little tendency to disperse. Reduction in salt concentration, however, even if the ratio of calcium to sodium remains high, produces a relatively lower permeability, apparently due to the increased dispersibility and reduced mean effective pore radii within the soil which result from a smaller electrolytic content in the water.

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[9] From data obtained in the laboratory of the Div. of Soil Technology, Univ. of California, by G. B. Bodman and M. Fireman.

The effect of the sodium-rich waters is distinct—still lower permeabilities are produced; but here again, although both waters have the same sodium-to-calcium ratio, the salt-concentration effect—due to repressed dissociation of the soil clays—is more pronounced in the water containing 5,000 ppm than in that containing only 500 ppm.

TABLE 10.—PERMEABILITY OF TWO REPRESENTATIVE SOIL TYPES AS AFFECTED BY PRELIMINARY TREATMENT WITH HIGH SODIUM WATER

Description	Yolo silty clay loam		Aiken clay loam	
Composition of Water Used During Preliminary Treatment:				
Ratio of milliequivalents of sodium to calcium	—	6 : 1	—	6 : 1
Concentration of sodium chloride plus calcium chloride, in parts per million	0	4,000	0	4,000
Permeability (Water-Saturated), as Inches per Day per One Gravity:				
During the 46th day of preliminary treatment	0.31	0.75	7.3	21
During the 10th day of "reclamation" with water containing 500 ppm of chlorides (Na : Ca = 6 : 1)	0.30	0.64	6.1	16

Upon examination of the conditions during the ensuing washing process with distilled water, it was found that the permeability was reduced to one tenth to one two-hundredth of its former value by removal of the excess salts. The sealing effect produced by the preliminary treatment with water containing 5,000 ppm of sodium chloride plus calcium chloride in the equivalent ratio of Na : Ca = 8 : 1 is most pronounced of all, and after 10 days of washing the column receiving this treatment had reached the low value of 0.03 in. per day under a driving force of one gravity. The entire series of washed soils possessed permeabilities in a predictable sequence. Chemical analyses of the leached soils from these particular trials have not been made; but results with other, similar experiments justify the expectation that, during the time period concerned, the high calcium water has produced a more calcium-rich soil at the expense of magnesium and sodium, and that the high sodium water has increased the exchangeable sodium in the soil about 500% to 1,000%, mainly at the expense of the magnesium. Complete replacement of existing cations by sodium has not been obtained under these conditions of leaching. Furthermore, the extent of alteration doubtless has been in proportion to the concentration of salts in the two chemical types of water.

Table 10 is presented primarily to illustrate the difference in behavior exhibited by two soils of rather similar apparent mechanical composition but very different clay constitution. The trials were made in a room maintained at a constant temperature of 30° C. The first, the Yolo silty clay loam, contains a highly siliceous clay of the montmorillonitic (bentonitic) type. The second, the Aiken clay loam, contains a much more ferruginous and aluminous clay of the kaolinitic type.

The former is potentially a much more favorable material than the latter from the engineer's point of view, in so far as providing an impermeable clay lining is concerned. The Aiken responds extremely poorly to any attempts to disperse it and render it retentive of, and impermeable to, water. Table 10 also illustrates the importance (if impermeability is desired) of removing excess salts by means of a relatively pure water. Note that the so-called "reclamation" process here consisted of washing with a water containing 500 ppm of high sodium water. The use of salt-free water would have reduced the permeability of the Yolo much more rapidly and completely, although as judged by other experiments not here reported it is extremely doubtful whether the Aiken, owing to the nature of its clay, could ever be reduced to the high degree of impermeability exhibited by the Yolo.

The Soil Technology Laboratory has been able to produce very low permeabilities in artificially packed columns of several different soils by long-continued leaching with pure water alone. The explanation seems to lie in the removal of the excess salts (originally present as the soil solution), dispersion, and subsequent local migration of clay, and in the marked reduction in the number of the coarser micropores in the soil. Hitherto unexplained increases in permeability, however, both to salt-free and saline water have repeatedly been obtained in the Soil Technology Laboratory, University of California, during long-continued percolation of water columns.

Finally, it should be emphasized that the entire permeability-time curve for a clay seal in a water reservoir, not only during its preparatory treatment by leaching, but also later during its actual use, will depend upon numerous factors. The permeability at any given time is partly controlled by the permeability up to that time, since it is the opportunity of exchange between ions at the surfaces of the individual clay particles and ions held in solution by the percolating water which is the essential preliminary to any alteration in base status of the clay and hence also to alterations in its physical behavior. The clay particles contained in the seal always tend to come to physicochemical equilibrium with the water solution by which they are bathed, so that an equilibrium distribution of ions between clay and liquid is approached, if not actually attained. Evidently after preparation of the seal, replacement with sodium, washing to remove electrolytes, and filling the reservoir with storage water, a fresh approach will be made toward a new physicochemical equilibrium between clay and solution (the storage water). Long-time storage of high-lime water, or of highly saline water, would not prove successful, and it seems probable that, even with water of a high degree of chemical purity, an occasional replenishment of sodium ions in the clay seal might be desirable at intervals. So far no experiments have been conducted continuously over a sufficiently long time to answer this latter question.

JOHN D. WATSON,[10] ASSOC. M. AM. SOC. C. E. (by letter).—The author states (see heading "Conclusions") that "this method of sealing would appear to have a wide field of usefulness in constructing impervious membranes for water works structures such as reservoirs and dams." Although the writer cannot deny the efficacy of the salt in ionizing the solution and thereby hastening the effectiveness of the clay seal, nevertheless it must be pointed out that if the clay had been properly applied, the salt would not have been needed.

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[10] Asst. Prof., Civ. Eng., Coll. of Eng., Duke Univ., Durham, N. C.

In the first place it was an error to buy a soil to be used for sealing the Lagoon on the basis of the particle sizes of the soil. The permeability of a soil can be indicated only very approximately by a mechanical analysis. The late Allen Hazen, M. Am. Soc. C. E., made a point[11] of this fact in 1924, but many engineers are still beguiled by the imagined magic of the grain size distribution curve. The best measure of imperviousness is the coefficient of permeability according to Darcy's law, and this coefficient can be determined very readily by a soils laboratory. The most desirable material to use in this case would have been the material with the lowest coefficient of permeability, provided it was not subject to excessive swelling which would cause an increase in its permeability. Other tests by the soils laboratory could have measured this factor of swelling.

After it has been dried, clay will slake when it is wetted again. The layer of clay in this Lagoon was subjected to two months of severe drying, and the author states that the entire layer was fissured with shrinkage cracks. It is no surprize, therefore, that when water was put into the Lagoon, the leakage was excessive and "the clay was found to have softened to a mush for a depth of 8 in. or more." Since the clay was allowed to dry out completely and was then wetted again, it slaked. This slaking accounts fully for the high rate of seepage and for the complete softening of the clay layer. The tests to determine the optimum moisture content, the sprinkling process, and the rolling were all so much wasted effort after the clay was allowed to dry out. Incidentally, why were the tests to determine optimum moisture content ever made at all, since "no effort was made to attain a condition of uniform moisture before rolling"?

The author points out that when salt water was put into the Lagoon "the clay, although softening to a depth of 2 in., was hard and cohesive for the remainder of its depth." The previously mentioned 8-in. depth of softening was with fresh water. The author does not state with equal emphasis that the 8-in. depth of softening was determined six days after fresh water was admitted to the Lagoon, whereas the 2-in. depth of softening was determined not less than sixty-five days after sea water was put in. The writer ventures the opinion that at the end of the first six days the clay was softened by the salt water to the same depth to which it had been softened by the fresh water. During this time interval of two months the clay was reconsolidated by a force which the author has not mentioned. Karl Terzaghi, M. Am. Soc. C. E., has called this force, produced by any fluid flowing through a porous medium, the "seepage pressure." It is simple to show by elementary mechanics that the magnitude of seepage pressure is equal to the product of the unit weight of the fluid, the hydraulic gradient, and the volume of the medium. With an 18-in. depth of sea water over a 10-in. layer of clay, the seepage pressure amounted to 150 lb per sq ft. Although this amount is insignificantly small when compared with any type of roller, it is nevertheless large enough to consolidate a mass of very soupy clay to a high degree of impermeability after a sufficient period of time. In this particular case 100% consolidation might have been reached in two months, although it is most unlikely that it could have been reached in six days.

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[11] *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 62.

The points which the writer has endeavored to make are these: (1) The clay should have been judged on the basis of its permeability rather than the sizes of its particles; (2) the clay should have been submerged before it was allowed to dry; and (3) even though the clay was allowed to dry, ultimately it would have sealed itself without the salt, although the salt was advantageous in hastening the process.

John W. Pritchett,[12] Assoc. M. Am. Soc. C. E. (by letter).—The wide dispersion of clay, its availability in large quantities, and the low cost of obtaining and handling it account for its increase in use and importance as a material for construction. With increased research and increased knowledge in the modes of testing and utilizing this material, there is being developed scientific knowledge upon which more definite conclusions may be reached as to the behavior of clays of various types and under various usages. The interesting paper by Mr. Lee is a distinct contribution to such knowledge. It brings to attention the importance of thorough analyses of clay materials proposed to be used in construction where imperviousness is important, and in localities where such types of clays are likely to be encountered. Whether or not such types are of common occurrence in a given locality is a matter that should receive the attention of those concerned with designing structures having clay as part of the structure or the foundation.

The practicability of using clay for the purpose of obtaining impervious lining for salt-water storage pits in part of the East Texas oil field has recently become a matter of importance. An area of about 100 sq miles in the southwestern part of this large oil field lies within the Neches watershed. On November 1, 1939, it was estimated that 1,567 oil wells in this field and within the Neches River watershed were producing salt water. The daily allowable production of oil in this area is 20 bbl per well. An estimate of salt water being produced in 1940 was 74,000 bbl per day. The maximum per well has been from 900 to 1,000 bbl per day. It has been estimated that at least 31 wells were each producing 450 bbl or more per day. Analysis of the water being produced from a representative well in this area is as follows (in parts per million):

Total solids	61,170.0	Iron	7.0
pH	8.2	Manganese, less than	0.02
Phosphorus alkalinity	0.0	Sodium	0.0
Total alkalinity	435.0	Carbonate	0.0
Silica residue	220.0	Bicarbonate	531.0
Turbidity	55.0	Sulfate	643.0
Fixed salts	58,630.0	Chlorides	35,500.0
Calcium	1,240.0	Fluoride	1.8
Magnesium	572.0	Nitrate, less than	0.4
Total hardness	5,443.0	Selenium	0.0
Sodium (calcium)	1,036.0		

The problem was to formulate some plan for the operation of oil wells in this area in such a manner that salt-water contamination of the Neches River could be abated sufficiently so that the quality would meet the requirements for municipal, industrial, and rice irrigation uses in the lower reaches of the stream, and would also not be harmful to fish life. As a part of this larger problem, tests were made on clay and asphalt samples which are interesting in so far as they apply to the results reported by Mr. Lee.

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[12] Member, State Board of Water Engrs., Austin, Tex.

TABLE 11.—RESULTS OF SOIL ANALYSIS

Description	Sample No.:													
	1	2	3	4	5	6	7	8	9	10	11	12	13	14
(a) PHYSICAL CHARACTERISTICS														
Cylinder No.	..	1	2	..	3	..	..	6	7	..	..	10	11	12
Density, wet, pounds per cubic foot	..	126	126	..	122	..	..	119	122	..	..	122	118	116
Density, dry, pounds per cubic foot	..	104	104	..	100	..	..	91	96	..	..	96	93	89
Percentage of water in mix	..	20	22	..	22	..	..	30	26	..	..	26	26	30
Soil classification [a]	S	..	C_l	S	C_s	S	S	C_l	C	S	S	C	C	C
(b) MECHANICAL ANALYSIS														
Colloids (%)	..	..	18	..	17	..	..	17	16	..	..	25	35	35
Clay and colloids (%)	..	..	28	..	28	..	..	27	35	..	..	46	85	57
Silt (%)	6	..	42	10	52	5	5	23	43	5	7	42	7	36
Fine sand (%)	85	..	25	27	19	75	17	38	17	50	57	10	8	6
Coarse sand (%)	9	..	5	13	1	20	78	12	5	45	36	2	0	1
Specific gravity	..	..	2.71	..	2.66	..	..	2.66	2.70	..	..	2.68	2.72	2.74
Porosity (%)	..	..	39.5	..	32.8	..	..	45.0	42.7	..	..	42.4	45.1	47.8
Voids ratio	..	..	0.652	..	0.488	..	..	0.817	0.747	..	..	0.736	0.820	0.914

[a] S = sand; C = clay; C_l = clay loams; and C_s = silty clay.

Asphalt Sample.—A sand-asphalt mixture pressed into a cylinder about 4 in. in diameter and 2 in. thick was submerged in a solution containing 40,000 ppm chlorides. This material is similar to that used for earth stabilizer on asphalt highway construction. After being submerged ten days or two weeks, only the slightest indication of deterioration was observed. The sample was then set aside, allowing the water to evaporate. After three months the sample was examined. It was found to be encrusted in a layer of salt crystals, but plainly showed cracks, indicating deterioration. On breaking away the salt crust, almost complete breakdown of the material was observed. The outer edges fell apart easily, and any part of the sample could be mashed and crumbled between the fingers.

Clay Sample.—For purposes of examination and test 14 samples of clay were secured. These samples were considered representative of the available clays in the southern part of the area where proposed storage pits would probably be constructed. The results of the soil analysis were as shown in Table 11.

Of the 14 samples secured, eight were mixed to putty-like consistency with (*a*) city tap water, (*b*) water with 10,000 ppm of chlorides, (*c*) water with 20,000 ppm of chlorides, and (*d*) water with 40,000 ppm of chlorides. Each mixture was rolled by hand into 2-in. balls and allowed to dry in air for 3.5 days, the purpose being to duplicate the tests made by the author, as described

TABLE 12.—TESTS OF SALT TREATMENT; SAMPLES SUBMERGED IN TAP WATER

No.[a]	DESCRIPTION		CONDITION AFTER SUBMERGENCE		
	Color	Surface	Two hours	Ten hours	Twenty hours
			(a) MIXED WITH CITY WATER		
2	Light chocolate	Smooth	Almost disintegrated	Disintegrated	Disintegrated
3	Light red	Lightly checked	Almost disintegrated	Disintegrated	Disintegrated
5	Cream	Smooth	Good condition	Disintegrated	Disintegrated
8	Light tan	Smooth	Lower half disintegrated [b]	Disintegrated	Disintegrated
9	Medium red	Smooth	Disintegrated	Disintegrated	Disintegrated
12	Grayish yellow	Smooth	Disintegrated	Disintegrated	Disintegrated
13	Red	Lightly checked	Lower half disintegrated [b]	Disintegrated	Disintegrated
14	Red	Smooth	Disintegrated	Disintegrated	Disintegrated
			(b) MIXED WITH CITY WATER CONTAINING 10,000 PPM OF CHLORIDES		
2	Darker chocolate	Granular	Almost disintegrated	Disintegrated	Disintegrated
3	Light red	Smooth	Slightly disintegrated, lower surface	Two thirds disintegrated [d]	One half disintegrated [d]
5	Darker cream	Smooth	Lower third slowly disintegrating	Three fourths disintegrated [d]	Disintegrated
8	Darker tan	Smooth	Lower surface slowly disintegrating	One half disintegrated [e]	One half disintegrated [d]
9	Medium red [c]	Smooth	Almost disintegrated	Disintegrated	Disintegrated
12	Darker yellow	Smooth	Upper fourth intact, but critical	Disintegrated	Disintegrated
13	Darker red	Lightly checked	Lower two thirds disintegrated	One third disintegrated [f]	One half disintegrated [e]
14	Darker red	Smooth	Lower two thirds disintegrated	One half disintegrated [f]	One half disintegrated [h]
			(c) MIXED WITH CITY WATER CONTAINING 20,000 PPM OF CHLORIDES		
2	Darker chocolate	More granular	Almost disintegrated	Disintegrated	Disintegrated
3	Darker red	Granular	Lower surface, continued disintegrating	One half disintegrated [d]	Two thirds disintegrated [d]
5	Darker cream	Granular	Intact; good condition	One half disintegrated [d]	Disintegrated
8	Dark tan [c]	Smooth [g]	Lower surface, continued disintegrating	Two thirds disintegrated [d]	Two thirds disintegrated [d]
9	Dark red	Smooth [g]	Almost disintegrated	Disintegrated	Disintegrated
12	Dark yellow [c]	Smooth [g]	Almost disintegrated	Disintegrated	Disintegrated
13	Darker red	Lightly checked [g]	Little disintegration; top stable	One third disintegrated [h]	Two thirds disintegrated [h]
14	Darker red	Smooth [g]	Lower two thirds disintegrated	One half disintegrated [d]	Disintegrated
			(d) MIXED WITH CITY WATER CONTAINING 40,000 PPM OF CHLORIDES		
2	Darker chocolate	Smooth	Disintegrated	Disintegrated	Disintegrated
3	Much darker red	Smooth	Firm; good condition	One third disintegrated [e]	Two thirds disintegrated [h]
5	Darker cream	Smooth [g]	Firm; excellent condition	One half disintegrated [d]	Almost disintegrated [d]
8	Much darker tan	Smooth [g]	Lower surface, continued disintegrating	Two thirds disintegrated [d]	Two thirds disintegrated [d]
9	Much darker red	Smooth [g]	Almost disintegrated	Disintegrated	Disintegrated
12	Much darker yellow	Smooth [g]	Almost disintegrated	Disintegrated	Disintegrated
13	Much darker red	Smooth [g]	Firm; good condition	One half disintegrated; cracked	Two thirds disintegrated [h]
14	Darker red	Smooth [g]	Firm; lower surface disintegrating	One third disintegrated [h]	Disintegrated

[a] Samples air dried 3.5 days. [b] Remainder firm. [c] No change in color. [d] Top shaped, but critical. [e] Top shaped and firm. [f] Top cracked; soft and critical. [g] Salt spots. [h] Top shaped; some stability.

under the heading "Tests of Salt Treatment." One full set of each mixture was submerged to two thirds the diameter in tap water for observation. Table 12 shows the results of these tests. Four of the samples were duplicated as per mix (*d*) (water with 40,000 ppm of chlorides), and after 3.5 days of air drying, were similarly submerged, but in water having 40,000 ppm of chlorides. Table 13 shows the results of this test.

In contrast with the results obtained by Mr. Lee, after submergence for two hours, the samples mixed with salt water showed nearly the same degree of disintegration as did the samples mixed with fresh water. After ten hours submergence most of the samples were either disintegrated or nearly so, whereas Mr. Lee states that his No. 3 and No. 4 showed no change in condition after two hours submergence.

In addition to the foregoing tests, 70 g of each of the eight clay samples shown in Table 13 were well broken down after being air dried and then covered to a depth of one-half inch with water containing 40,000 ppm of chlorides. Three and one-half days later the surplus water was drained off and the samples were allowed to dry in the air to a putty-like consistency, at which stage none of them appeared to have lost any noticeable degree of cohesion. This material was then rolled into balls and allowed to dry. After three months the balls were hard like baked clay, with no cracks or signs of deterioration.

TABLE 13.—TESTS OF SALT TREATMENT; SAMPLES SUBMERGED IN SALT WATER

Sample No.[a]	CONDITION AFTER SUBMERGENCE FOR:		
	Two hours	Ten hours	Twenty hours
2	Disintegrated	Disintegrated	Disintegrated
3	One half disintegrated [b]	Disintegrated	Disintegrated
9	One third disintegrated [b]	Three fourths disintegrated [d]	Four fifths disintegrated [b]
13	Slowly disintegrating [c]	One half disintegrated [c]	Two thirds disintegrated [e]

[a] Samples air dried for 3.5 days. [b] Top shaped, but critical. [c] Top firm. [d] Top shaped, but plastic. [e] Top shaped; some stability.

It was desirable to determine whether or not the clay samples, if used as lining for salt-water storage pits, would lose their cohesive properties if submerged continuously in water having a chloride content as high as that produced from the oil wells. The results of the investigations indicated that under proper methods of construction, several of the clays analyzed might be expected to produce a tank lining sufficiently impervious for all practical purposes. Tests on such samples taken from cylinders prepared in the laboratory, moulded under a pressure of 100 lb per sq in., showed a permeability of less than 0.025 or 1/40 of an inch per day per foot of head per foot of thickness. Chemical analyses of the soluble contents of the clay samples were not obtained.

Many storage tanks previously constructed in this area were found to be losing considerable water through seepage. Newer tanks have clay linings, with observation wells at each tank. Inspections will be made to determine whether or not excessive seepage losses occurred.

The problem of providing an impervious lining for pits used for the storage of salt water is somewhat different from the problem presented by Mr. Lee where salt treatment was used to obtain a more impervious lining for a reservoir for the storage of fresh water. The lining of the salt water storage pits would not be subjected to fresh water treatment after the introduction of salt water. Just what effect the continuous exposure to salt water will have on the permeability of the lining is a matter of much interest. Mr. Lee's paper brings to attention the importance of giving due consideration to the effect of salt on clay materials proposed for construction uses.

CHARLES H. LEE,[13] M. AM. SOC. C. E. (by letter).—Although he had hoped that the experience described in his paper would be more widely discussed, the writer is appreciative of the high quality of the three contributions submitted.

Professor Bodman draws from his extensive experimental work with agricultural soils in Western United States to develop the physicochemical background of the sealing action of salt. Of special interest to engineers is his statement of the relative adaptability to salt treatment of soils with similar mechanical analysis but with greatly differing clay constituents. He shows that highly siliceous clays of the montmorillonitic or bentonitic type can be rendered impervious much more readily than those of the kaolinitic type, which contains much greater percentages of iron and aluminum. The clay used in the lining of the Lagoon at Treasure Island was of the siliceous type.

In passing, it is to be noted that this chemical classification of clays refers to the clay particles that act as negatively charged cores or anions, whereas the writer's classification as "calcium" or "sodium" clay was with reference to the surrounding and much smaller positively charged cations. Both classifications are required for a complete chemical discription of a clay.

With reference to the permanence of a reservoir lining rendered impermeable with salt, Professor Bodman states that occasional replenishment of sodium ions might be necessary to maintain impermeability, not alone with respect to highly calcareous or saline waters, but even for water of low mineral content. Water used to fill and replenish the Lagoon throughout the life of the Exposition was obtained from the San Francisco Water Department. Typical analyses of this water at various times of the year indicate a water of low mineral content, with calcium predominant (see Table 14).

During twenty-one months of service, the Lagoon showed no tendency to lose its initial seal. It is unfortunate that the Lagoon must be destroyed because longer service would have furnished valuable information on this point.

Mr. Pritchett's contribution to the discussion is of especial interest as it illustrates, strikingly, two points of great importance in the use of salt as a sealing agency. To begin with, the predominating soils in the region where Mr. Pritchett is working are of the red and yellow podsol type composed of lateritic material with a "silica sesquioxide ratio" of approximately 2. Clays associated with this type of soil are of the kaolinitic type. Extensive areas of this soil occur in eastern Texas, western Louisiana, southern Mississippi and

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[13] Cons. Engr., San Francisco, Calif.

Alabama, and central Georgia where it has been termed the Norfork-Ruston soil series by the Bureau of Soils, U. S. Department of Agriculture.[14] Color description of clay test samples listed in Table 12 show a predominance of reds and yellows, and it is probable that tests were all made on clays of this type.

TABLE 14.—MINERAL ANALYSES, SAN FRANCISCO WATER DEPARTMENT (Parts per Million)[a]

Date (1937)	Solids[a]	Si	Na	Ca	Mg	Fe	Al	HCO_3	CO_3	SO_4	Cl
February 10[b]....	34.0	2.9	9.4	10.9	4.2	0.3	0.25	56.0	0	7.2	8.0
May 11[c]........	91.0	6.7	7.0	13.1	5.3	0.19	0.49	61.0	0	11.7	9.0
November 15[c]...	124.0	2.9	11.0	19.4	7.6	0.1	0.26	90.0	0	15.6	11.0

[a] Solids = dissolved solids; Si = silica and insoluble matter; Na = sodium (including potassium, K); and Fe = iron (dissolved); other columns are as shown. [b] Sample from the surface of Crystal Springs Reservoir. [c] Sample from the water line at Millbrae, Calif.

As stated by the writer, and also by Professor Bodman, sodium treatment has little effect upon the physical properties of such clays. This is probably the explanation for the failure of the salt admixture to produce a degree of disintegration in clay balls when submerged in water, differing from that of balls mixed with fresh water.

The other point involves the proposed storage of salt water in reservoirs sealed with salt-treated clay. The writer's experience at Treasure Island was that less than 50% of the final sealing effect was obtained by use of salt water alone, the fresh water being necessary to leach out excess sodium before the full effect of salt treatment was obtained. Laboratory tests also show the necessity of fresh-water leaching following salt application before permeability is effectively reduced. Professor Bodman states that "Long-time storage of high-lime water, or of highly saline water, would not prove successful." Even if a satisfactory clay were found in local deposits of alluvial materials along the Neches River, it is doubtful whether salt treatment would solve the problem of storage of such highly saline waters as are encountered in the east Texas oil field. Compaction or some method of soil stabilization would seem to be the only answer in this region.

In opening his discussion, Professor Watson states that "* * * if the clay had been properly applied, the salt would not have been needed." The writer is in entire accord with this statement, assuming that "properly applied" refers to moisture control and method of rolling. Acceptance of the clay by the writer was predicated upon placing with uniform moisture at optimum moisture content and rolling with a sheepsfoot roller. Neither requirement was followed in actual construction, however, for reasons beyond the writer's control. It is the writer's opinion that a sheepsfoot roller could have been used satisfactorily to compact the clay layer, without breaking through into the sand, either by increasing the thickness of the first clay layer or by stabilizing the sand subgrade by sprinkling it with water before spreading the clay. As for moisture control, its use is now almost universal in this type of construction, and methods for its accomplishment are a matter of routine. The salt treatment was prescribed only after the completion of the work, when a test of the poorly compacted clay membrane disclosed its leaky condition.

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[14] "Soils and Men," *Yearbook of Agriculture*, U. S. Dept. of Agri., 1938, p. 1069 and inset map.

The writer does not agree fully with Professor Watson's statement that it was an error to buy a soil on the basis of particle size. Although the specifications, as prepared by the Construction Division of the State Department of Works, might well have provided for a permeability test, such test would have been subject to the same limitation as was the mechanical analysis—namely, the failure of the clay material, as actually placed, to conform in density with the laboratory test samples at ideal density. The writer has directed permeability tests for many years, using Darcy's law, and has found wide differences in the same material with differing degrees of density.

With regard to Mr. Hazen's precautionary statement, referred to by Professor Watson, careful reading of the original discussion shows that he had in mind the indiscriminate use of mechanical analysis in conjunction with the Hazen formulas for flow of water through filter sands and other clean granular materials of uniform grain size. He states that these formulas and their constants were developed for a special type of filter sand and do not apply to the general run of natural earth materials used in engineering construction.

In another paper Mr. Hazen states,[15]

> "It is to be noted that the formula was never intended to apply to clays, hardpans, soils, and other materials. The effort to apply it to such materials is not to be encouraged, and the results are not to be depended on."

It was not the mechanical analyses, as such, that he warned against, but the indiscriminate use of his formulas which, as a preliminary step, involved mechanical analysis.

In discussing these matters with Mr. Hazen many years ago, in connection with the computation of underflow through the narrows of an alluvial-filled river valley, the writer also recalls that he advised against the application of his formula to natural alluvial materials but did not advise against mechanical analysis as it might throw light upon permeability as affected by distribution of particle sizes. In fact, a very important element in the Hazen formula is the constant "c" which has wide variation, depending largely upon the variations in proportions of large and small particles as expressed by the uniformity coefficient obtained from the results of mechanical analysis.

It has been the writer's experience that those natural earth materials whose distribution of particle sizes conforms closest to the general grading equation

$$P = \left(\frac{d}{D}\right)^n \qquad \text{.........................(1)}$$

(in which n lies between 0.25 and 0.40) are the most impermeable, and that material substantially following this equation with a degree of fineness sufficient to include at least 3% of clay, or poorly graded material containing at least 25% of clay, when thoroughly compacted, is practically, if not fully, watertight.[16] Mechanical analyses of samples submitted by the contractor indicated that the selected clay fulfilled requirements of the grading formula, Eq. 1, and personal handling of the clay by the writer gave added confidence as to its impermeability if properly compacted.

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[15] *Transactions*, Am. Soc. C. E., Vol. LXXIII, September, 1911, p. 201.

[16] "Selection of Materials for Rolled-Fill Earth Dams," by Charles H. Lee, *Transactions*, Am. Soc. C. E., Vol. 103 (1938), p. 1.

The writer's statement of conditions during the two months' period between the completion of the placing of the clay membrane and the first flooding is probably incomplete, as Professor Watson states that the clay "was subjected to two months of severe drying, and * * * the entire layer was fissured with shrinkage cracks." As a matter of fact, the entire surface of the clay was sprinkled once each day throughout the two-month period. The rate of evaporation, augmented by afternoon trade winds, was so great, however, that the surface dried out before the day was over and thus gave opportunity for cracking. Serious cracking was limited to the top layer, cracking in the bottom layer being superficial. The writer does not agree with the statement that the "* * * slaking accounts fully for the high rate of seepage and for the complete softening of the clay layer." It certainly was a contributing factor, but the basic cause was inadequate compaction. The daily sprinkling would probably have been adequate if the material had been compacted to the density possible at optimum moisture content. It had been planned to flood the clay layer immediately after completion, but unforeseen delay in the execution of another lagoon contract made this impossible.

With regard to the relative rate of downward progress in softening of the clay with fresh and with salt water, tests were made during the first six days of submergence by salt water, and softening did not occur to the same depth as with fresh water. In fact, the depth of softening at the sixth day was 2 in., or less, and was not materially greater at sixty-five days. The rate of compaction under a seepage pressure amounting to 1 lb per sq in., as computed by Professor Watson, is so slow that many years would have been required to harden the clay, rather than a few weeks, as he suggests. Under certain conditions, sealing of leaky reservoir bottoms by seepage pressure, by accumulation of a skin layer of fines, or by other means, has been experienced by the writer. Such sealing, if it occurs, requires a period of years for its consummation, whereas expositions come and go within a period of months. Application of effective salt treatment requires only a few weeks. In practical engineering the means for accomplishment must be adapted to the end in view. The engineer must select those forces of nature which will meet schedule as well as be effective.

Returning to Professor Watson's opening statement, he questions the utility of the salt method in view of the possibility of securing impermeability by compaction. Although it is true that impermeability of a well-graded material can be obtained by compaction, this is not so true of clay materials lacking the sand fractions. For such materials, for the reduction of hazard with compacted clay materials having the sand fractions, and for remedial work such as that described by the writer, the salt method unquestionably has a future.

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CHICAGO RIVER CONTROL WORKS

By H. P. Ramey,[1] M. Am. Soc. C. E.

With Discussion by Messrs. A. E. Niederhoff, R. P. V. Marquardsen, Isaac DeYoung, John W. Woermann, Edward Soucek, W. C. Weeks, Henry R. King, and H. P. Ramey.

Synopsis

The Chicago River Control Works constructed in 1939 by The Sanitary District of Chicago (Ill.) prevent the polluted Chicago River from entering Lake Michigan. They consist of walls making a watertight enclosure around

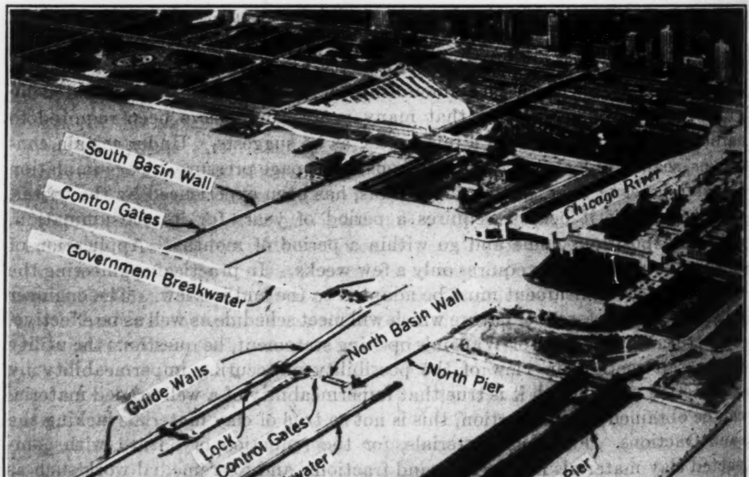
South Basin Wall
Control Gates
Government Breakwater
Chicago River
North Basin Wall
North Pier
Guide Walls
Lock
Control Gates
Breakwater
Navy Pier

Fig. 1.—General View of Chicago River Control Works

Note.—Published in January, 1940, *Proceedings*.

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the river mouth, a navigation lock to pass shipping, and two sets of control gates to regulate the quantity of water admitted into the river from the lake (see Fig. 1). The westerly or river end of the lock is approximately 1,700 ft east of the Outer Drive Bridge, which crosses the Chicago River at its mouth. The longitudinal axis of the lock extends from east to west, in line with the river. The south wall of the lock is extended 700 ft easterly of the east lock gate, to serve as a guide wall and breakwater. From the northwest corner of the lock a north basin wall runs north, connecting with an existing United States North Pier which in turn connects with the shore. In this wall are four control gates. From the southwest corner of the lock a guide wall extends westerly and slightly southerly, connecting with the northerly end of the United States breakwater. This breakwater extends due south, and from a point 1,600 ft south of its connection with the guide wall, a south basin wall was constructed to the west, connecting this breakwater with the shore. Four other control gates are in this south basin wall. The two basin walls, the lock, and the southwesterly guide wall, in conjunction with the original United States breakwater and North Pier, enclose an inner harbor basin about 1,700 ft by 2,000 ft.

Many novel features were embodied in the design of the lock gates and walls and these are discussed herein, as well as the purpose of the Chicago River Control Works.

History

The purpose of the Chicago River Control Works is to separate, definitely and effectively, the Chicago River from Lake Michigan, which is the source of municipal water supply for Chicago and its environs. A detailed account of the history of this project has been presented by Samuel N. Karrick, M. Am. Soc. C. E., Major, Corps of Engineers, U. S. Army.[2]

General Conditions

In this paper all elevations are referred to Chicago City Datum (C.C.D.) which is 579.94 ft above mean sea level. The average level of Lake Michigan is about 581.00 ft above mean sea level, or about + 1.00 C.C.D. Its highest level, monthly mean, was 584.60 in 1838, and its lowest 577.35 in January, 1926. This indicates a range of 7.34 ft. The level of Lake Michigan averages about 1.00 ft higher during the summer than in the winter. It averaged about El. − 1.50 C.C.D. during the construction seasons of 1936 and 1937.

The bed of the lake in Chicago Harbor at the site of the lock (see Fig. 2) has an elevation of about − 25.0 C.C.D. Soil tests, made by driving test piles, indicated soft blue clay for a depth of 20 to 30 ft below the lake bed, or from − 25.0 C.C.D. down to El. − 45.0 to − 55.0 C.C.D., where stiff blue clay was encountered.

The lock walls and harbor enclosure walls were designed to sustain a head of 8 ft of water in either direction. The river level may be higher than the lake, during flood conditions, but ordinarily the lake level will be the higher. In the future it may be desirable to hold the river level at some fixed elevation, such as − 2.00 C.C.D., to provide definite vertical clearance under bridges across the Chicago River, under which conditions the differential in water levels may average 2 ft. At present this differential is about 6 in., or less.

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[2] "Protecting Chicago's Water Supply," by Samuel N. Karrick, *Civil Engineering*, September, 1939, p. 547.

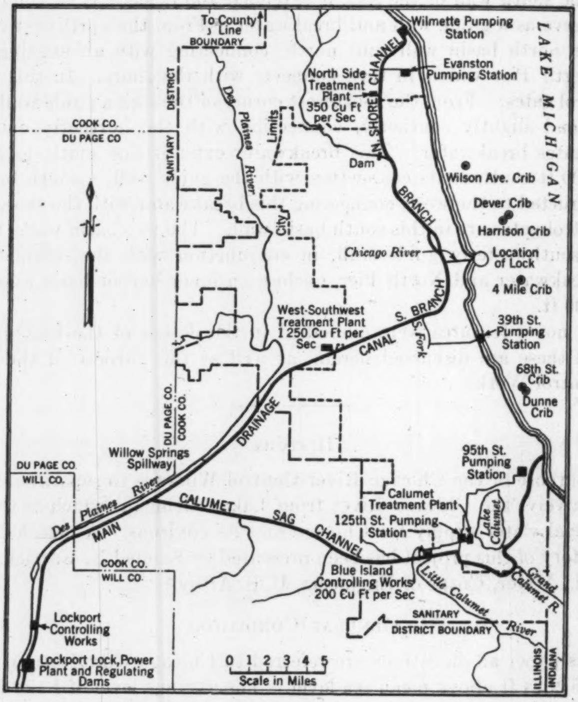

To County Line
BOUNDARY
SANITARY DISTRICT
COOK CO.
DU PAGE CO
N
Des Plaines River
Limits
City
Wilmette Pumping Station
N. SHORE CHANNEL
Evanston Pumping Station
North Side Treatment Plant 350 Cu Ft per Sec
Dam
LAKE MICHIGAN
Wilson Ave. Crib
BRANCH
Dever Crib
Harrison Crib
Chicago River
Location of Lock
4 Mile Crib
West-Southwest Treatment Plant 1 250 Cu Ft per Sec
S. BRANCH
S. Fork
CANAL
39th St. Pumping Station
68th St. Crib
Dunne Crib
DRAINAGE
DU PAGE CO.
COOK CO.
Willow Springs Spillway
DU PAGE CO.
WILL CO.
95th St. Pumping Station
Des Plaines River
MAIN
CALUMET
SAG
CHANNEL
Calumet Treatment Plant
125th St. Pumping Station
Lake Calumet
COOK CO.
WILL CO.
Blue Island Controlling Works 200 Cu Ft per Sec
Little Calumet River
Grand Calumet R.
SANITARY DISTRICT BOUNDARY
Lockport Controlling Works
Lockport Lock, Power Plant and Regulating Dams
0 1 2 3 4 5
Scale in Miles
ILLINOIS
INDIANA

Fig. 2.—Intake Cribs and Channels, Sanitary District of Chicago

It was necessary that the lock gates sustain a head of water in either direction, and that they remain closed, against surges and heavy seas from the lake, even when the static head might be very small. This eliminated the use of ordinary miter gates.

It was desirable that the control gates be stable, at any width of opening, in order to obtain reasonably close control of the flow from the lake into the river. It was desirable also that gates be installed as flood gates, to close automatically and prevent any flow whatever from the river into the lake so as not to place any reliance upon human judgment as to when the gates should be closed in case the river level became higher than the lake. This problem was solved by installing sluice gates for control, and flap gates on the inner side of the gate structures for protection.

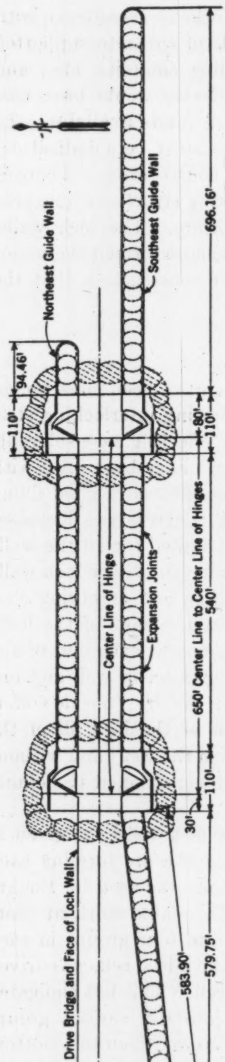
To have constructed the Chicago River Control Works in accordance with designs ordinarily used for such structures (lock walls of concrete supported on piles, miter lock gates, standard lock valves, filling conduits, etc., and connecting walls of rock-filled piers capped with concrete) would have cost about $4,800,000 or 60% more than the $3,000,000 of funds available. To effect a 60% saving in cost, it was obvious that there must be a radical departure from the orthodox method of lock and wall construction. There is no known way to construct lock gates, or their supporting structures, inexpensively. Such construction must be permanent in nature. The lock walls, guide walls, and the walls connecting the lock with the shore formed the major part of this particular job and it was in wall or pier construction that the principal saving was effected.

WALLS

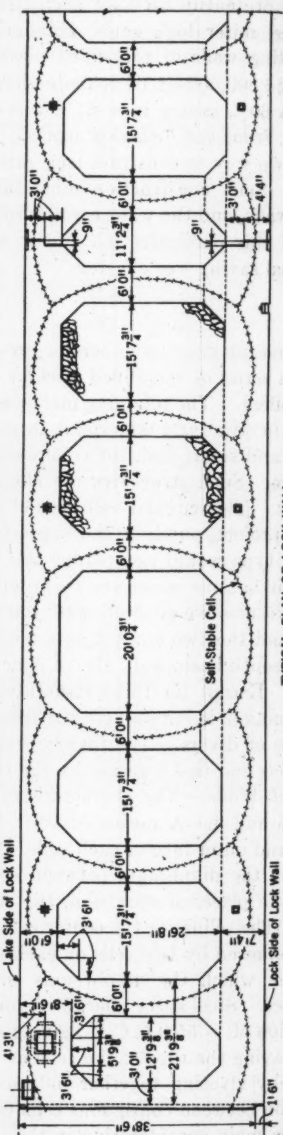
In recent years, numbers of breakwaters and piers have been constructed using a series of rock-filled circular cells formed by driving interlocking steel sheet piling. The retaining members are all in tension, effecting a considerable saving in structural material. In many places such cells have been filled with gravel and sand; and, for cofferdams so built, clay has proved a good filling medium. Such structures are slightly flexible and a concrete cap increases rigidity. This circular cell type of construction was adopted for all the walls in this project, purely in the interest of economy (see Fig. 3). Since lock walls of this type would not permit the construction of filling and emptying conduits, it became necessary to provide such facilities at the ends of the lock. The two massive concrete gate foundation structures, one at each end of the lock, and the two control gate sections, one in the north basin wall and one in the south basin wall, also of concrete, were built "in the dry," inside cofferdams. Except for these structures and for the floor of the lock all of the construction, being wall construction, was performed "in the wet" and without the use of divers. The four concrete structures were constructed in definite locations and tied together by the cellular type walls.

Lock Walls.—The characteristic dimensions of the lock walls are given in Figs. 3 and 4. A radius of 19 ft 2¼ in. was used for the arc forming each inner and outer face of each cell. A radius of 34 ft 1 in. was used for the arc forming the diaphragm between contiguous cells. To enable work at more than one place on each wall, the cells were divided into four groups in each wall. The piling for the center cell of each group, or the initial cell, was driven first, followed by two cells on each side of the center cell. This left elongated cells, in which the diaphragms curved inward, to join the various groups together. Steel sheet piles, 58 ft long, were used and were driven to a bottom elevation of − 55.0 C.C.D., giving a minimum penetration of 30 ft in the clay and leaving the tops of the cells at + 3.0 C.C.D. Special **Y**-piles were fabricated by riveting together half sections of regular sheet piles to form the junction between contiguous cells (see Fig. 3).

The main consideration in the steel sheet piling, as used, being always in tension, was strength in the interlocks. A steel was specified soft enough to


N
Northeast Guide Wall
Southeast Guide Wall
696.16'
94.46'
110'
80'
110'
Center Line of Hinge
Expansion Joints
650' Center Line to Center Line of Hinges
540'
110'
30'
583.90'
579.75'
26.6'
Center Line of Outer Drive Bridge and Face of Lock Wall
40'
192.78'
Copper Plug
4.66'

ARRANGEMENT OF STEEL SHEET-PILE CELLS, ALSO OF COFFERDAM CELLS FOR GATE BLOCKS


6'0"
15'7 3/4"
3'0"
3'0"
4'4"
9"
9"
11'2 3/4"
6'0"
15'7 3/4"
6'0"
15'7 3/4"
6'0"
20'0 3/4"
Self-Stable Cell
6'0"
15'7 3/4"
6'0"
15'7 3/4"
3'6"
6'0"
26'8"
7'4"
Lake Side of Lock Wall
Lock Side of Lock Wall
6'0"
3'6"
4'3"
5'9 3/8"
12'9 3/8"
21'9 3/8"
3'0"
3'6"
1'6"
38'6"
40'0"

TYPICAL PLAN OF LOCK WALL

FIG. 3.—PLAN OF CELLULAR TYPE WALLS

withstand the strain and distortion due to driving but hard enough to maintain rigidity once it was in place. The web thickness was $\frac{3}{8}$ in. and the width from point of interlock to point of interlock 16 in., with a tolerance of $\frac{1}{8}$ in. The phosphorous content and the sulfur content were each limited to 0.06% maximum and the tensile strength to a minimum of 70,000 lb per sq in. Bend test specimens were required to withstand bending 180° around a pin with a diameter twice the thickness of the specimen, without fracture on the outside of the bend. Three tests of pile sections were made of each heat of steel and the average interlock strength of the three tests was required to be not less than 18,000 lb per lin in., with no one test showing an interlock strength of less than 15,000 lb per lin in.

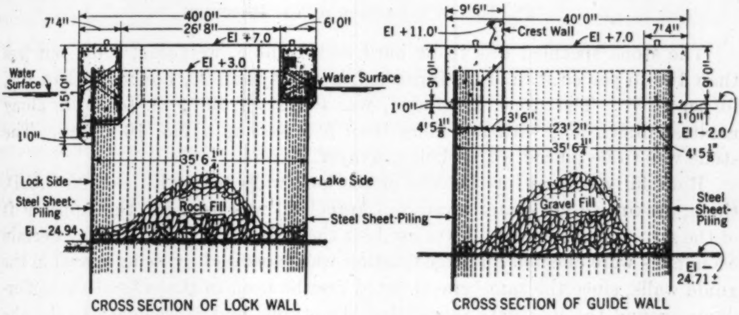

7' 4"
40' 0"
26' 8"
6' 0"
El +7.0
El +3.0
Water Surface
15' 0"
Water Surface
1' 0"
35' 6 1/4"
Lock Side
Lake Side
Steel Sheet-Piling
Rock Fill
Steel Sheet-Piling
El −24.94
CROSS SECTION OF LOCK WALL
9' 6"
40' 0"
El +11.0'
Crest Wall
El +7.0
7' 4"
9' 0"
9' 0"
1' 0"
3' 6"
1' 0"
4' 5 1/8"
23' 2"
El −2.0
35' 6 1/4"
4' 5 1/8"
Gravel Fill
Steel Sheet-Piling
El −24.71 ±
CROSS SECTION OF GUIDE WALL

FIG. 4.—CROSS SECTIONS OF CELLULAR-TYPE WALLS

No failure of any pile occurred in the permanent work. During construction, one failure occurred in a temporary **Y**-pile forming the junction between two inner walls of two cells of the cofferdam around the west gate foundation structure of the lock. Failure occurred in the web of the pile and was ascribed to exceptionally great stress caused by the use of a long radius in the arc of the inner curved surface of the cofferdam cell and the omission of weep holes in the piling. The cofferdam cells had no concrete cap. The radius used in this cell was 34 ft, compared to a radius of 19 ft $2\frac{1}{4}$ in. used for the cells of the lock wall. The maximum radius allowable was 28 ft. The accident was really caused by a boat striking the outer wall of the cofferdam.

The cells were filled, to El. − 2.0 C.C.D., with broken rock, graded in sizes from $\frac{1}{4}$ in. to 20 in. From this level, to El. + 7.0 C.C.D., the top of the walls, the structure was filled with gravel, 95% of which would pass a $\frac{3}{4}$-in. screen.

Rock was preferred as a medium for filling the cells, principally because of the shear that could be developed in a pile of stone, compared to that of any other medium. Clay would have been least expensive, as it was at hand, but it would have developed no shear and it could dissolve and escape through any hole or fissure. Sand would have assumed its final position quickly but it would have developed no shear and fine sand could have escaped through a hole or fissure. Gravel had the quality of assuming its final position quickly but of developing no shear. Stone had one drawback—namely, that it would gradually settle into the soft clay inside the cell at the bottom. This settlement was considered less serious than the faults inherent in the other materials. Alternate bids were taken for gravel fill and rock fill and the successful bidder submitted a price of $1.80 per cu yd for gravel fill, and $2.25 per cu yd for rock fill.

The gravel specified was to have at least 30% igneous material. It would have been of a quality suitable for use in concrete. The sizes were:

95% to 100% passing a ¾-in. screen
80% to 95% passing a ½-in. screen
15% to 20% passing a ¼-in. screen
0% to 5% passing a No. 10 screen

The stone specified was to be hard and durable, weighing, solid, not less than 160 lb per cu ft, and, as furnished, not less than 95, nor more than 105, lb per cu ft. The stone, as graded, was to vary from ¼ in. to 20 in. along greatest dimension, with not more than 5% passing a No. 10 screen. The stone was to be placed with a belt conveyer.

Rock fill was used for the cells of the lock walls below El. − 2.00 C.C.D. the approximate low-water level, and gravel fill was used for the upper 9 ft of the structure. Gravel fill was used for the cells of the guide walls. Certain economies were effected and construction was expedited by using gravel in the guide walls, since the same gravel could first be used in the cells of the cofferdams around the lock-gate foundation structures, and it was easy to dig the gravel from the cofferdams. These cofferdam cells also furnished convenient storage bins for gravel for concrete.

A concrete cap was constructed on top of the cells, formed of a grid of heavy concrete beams, each beam embedding the upper part of a cell face or a cell diaphragm. This concrete cap extended from El. + 7.0 C.C.D., down 16 ft, to El. − 9.0 on the inside of the lock, and down 10 ft to El. − 3.0 on the outer or lake side of the wall. Inside the cells, this concrete extended down 13 ft on the lock side of the cells, and 9 ft at the diaphragms and at the lakeward side. The width of the beam forming the face of the lock was 7 ft 4 in., most of the other beams being 6 ft wide. Holes were burned through the webs of the sheet piles where they were embedded in concrete, to permit the placing of reinforcing rods to tie together the bottoms of the concrete beams where there was a separation due to the sheet piling.

This system of concrete beams, forming a grill laid down over the rock-filled cells, provided a walk 7 ft 4 in. wide along the inner side of the lock wall and a walk 6 ft wide along the outer side, with cross walks 6 ft wide. This left open spaces over each cell, 26 ft 8 in. by 22 ft 8 in. over the initial cells and 26 ft 8 in. by 15 ft 7 in. over the remaining cells. Gravel, filling the upper portions of the structure, was leveled off at El. + 7.0 in these open spaces. Originally, a concrete slab, with several manholes through it, was planned, in place of these open spaces, to give a better footing than the loose gravel. Some settlement of the rock and gravel fill inside the cells is expected and when that has occurred these open spaces will undoubtedly be covered with a suitable floor. Three 1-in. mastic expansion joints were provided in the concrete cap of each lock wall, or at average intervals of 135 ft.

The advantage of the open concrete grill on top of the rock-filled cells, acting as a horizontal truss, over the same grill stiffened by a concrete slab is not apparent to many structural engineers. The difference in action, if any, should be very slight.

The lock wall was designed as a gravity dam, to sustain a head of 29 ft of water, a condition possible only if the lock should be empty and the water level on the outside at El. + 4.0 C.C.D. In addition, a horizontal thrust of 5,500 lb per lin ft of wall was assumed to be applied at El. + 5.5, to equal the force of a 6-ft wave. It was assumed that the cell would be drained empty of water by weep holes through the piling on the lock chamber side but that seepage through the lake side would decrease stability, to equal which, uplift was computed as a triangle at zero on the lock side and half the hydrostatic pressure on the lake side. Rock and gravel fill was computed at 100 lb per cu ft and the concrete cap at 150 lb per cu ft, these supplying the vertical downward force. Friction between the rock fill and the sheet piles, pulling resistance of the lake-side piles and supporting resistance of the lock-side piles were all disregarded. These assumptions resulted in the following total forces per linear foot of wall:

Direction	Forces, in pounds
Vertical downward	131,400
Vertical upward	15,800
Horizontal	32,900

The resultant of these forces falls slightly within the middle third of the assumed 34-ft wall base at El. − 25.0 C.C.D.

The horizontal pressure inside the cell at the base, due to the rock or gravel fill, was estimated at 2,240 lb per sq ft, with the lock empty and with no water in the cell. The direct unit tensile stresses in the steel sheet piles were computed as:

Location	Tensile stress
In the web (pounds per square inch):	
Of lock side sheet piles	9,600
Of diaphragm sheet piles	11,700
Of lock-side fabricated Y-piles	13,400
Of diaphragm fabricated Y-piles	16,200
In the interlock (pounds per linear inch):	
Of lock-side piles	3,580
Of diaphragm piles	4,300

The sheet piles were assumed to carry the entire weight of the reinforced concrete cap, producing the following vertical loads on the piles:

Type of pile	Load, in pounds per pile
Lock side	21,100
Lake side	13,700
Diaphragm	13,400

The supporting power of one pile, with a skin friction of 500 lb per sq ft and 30 ft of penetration, would be 40,000 lb.

No particular difficulties were encountered in the construction of these lock walls. A template was floated in the water, around which the piles were driven. Since the cells were formed in groups, work could be done at several places at one time. Because of possible distortion of the cells, when empty, by wave action, it was desirable to fill them with stone as quickly as possible. It was always advisable to have a cell empty when piling was being driven for its contiguous cell. The first cell was then more flexible and more easily permitted the driving of the final closure pile. The exposed portion of the steel sheet piles is permanently below water level, the top portion being encased in concrete down to the low-water line. The use of copper bearing steel for the piles was considered but the additional protection, in increased length of life of the piling, was not deemed worth the added expense.

Protection of Lock Walls.—No protection was given to the concrete in the walls of the lock from impact or rubbing by boats. Horizontal wales of either steel or wood were suggested, to be attached to the concrete inner faces of the lock walls, but were ruled out, one argument being that the edge of a small boat might get caught under a wale and the boat tilted. A suggestion that wooden wales be used to cap a row of wooden piles along each of the inner sides of the lock walls met with no favor because this would have decreased the usable width of the lock by approximately 2 ft. Floating fenders were also considered.

Use of the lock for less than one season has demonstrated the need for more care on the part of operators of boats or of some protection to the concrete if spalling is to be avoided. Large boats striking the walls have spalled off considerable pieces of concrete, particularly at the angle between the top and inner face of each wall. There has also been considerable spalling at the sides of certain vertical ladder recesses. None of the spalling has penetrated to any great depth and the stability of the walls has not been, and will not be, in any way impaired, but ultimately a ragged appearance will be created unless the impact of boats is avoided in some manner. It is largely a matter of judgment as to whether the expense of protective devices is justified or whether it is better to assume the cost of repairs.

Guide Walls.—Three guide walls were provided, using the same design and dimensions as for the lock walls, except that the concrete cap was extended down only 10 ft, to El. − 3.0 C.C.D., on both sides of the guide walls, or similar to the outer side of the lock walls.

The northeast guide wall is 94 ft long and serves only to aid in steering incoming boats into the lock. The southeast guide wall is 696 ft long and serves as a breakwater, as well as for mooring purposes. The southwest guide wall, 613 ft long, also serves as one of the enclosing walls of the inner harbor basin.

No special design was made for the guide walls. They are subject to impact from vessels in motion and to impact from waves, and a reasonable wall width was necessary. The width of 40 ft, as used in the lock walls, was maintained for uniformity as well as for ample stability.

For the most part, the easterly guide walls were constructed with sheet piling that previously had been used in forming the cofferdams around the concrete gate block structures during construction. Practically all of the cells of the guide walls were filled with gravel which previously had been used in the cofferdams around the gate blocks.

North Basin Wall.—The north basin wall is 216 ft 6 in. long, from the north face of the westerly gate block of the lock to the Government north pier, including the control gate section 59 ft long. The part of this wall of the cellular type is 157 ft long, and its design and construction is similar to that of the guide walls. The cells were filled entirely with broken rock ranging in size from ¼ in. to 20 in.

The reinforced concrete cap of the north basin wall is similar to the concrete grid over the guide walls, with the addition of a 24-in. solid concrete slab over the entire top, 40 ft wide, poured integrally with the concrete frame or grid. This also serves as the roadway approach to the lock, and a traffic lane, 9 ft wide, is kept clear of any obstruction.

South Basin Wall.—The south basin wall is 1,181 ft long, from the shore of Lake Michigan to the Government breakwater, including a concrete section, 59 ft long for four control gates. The water depth averages less than 20 ft and the width of the wall is 24 ft. This wall is of the straight-faced type, being formed of two rows of steel sheet Z-piling, retaining a rock fill. A stretch of 457 ft of this wall was constructed of steel sheet piles 47 ft long and the remainder of piles 55 ft long. All piles were driven until a uniform top elevation of + 5.75 C.C.D. was attained, giving an average penetration in the soil of about 25 ft.

A cap, formed of a 15-in. by 33.9-lb channel, was welded to the top of each row of Z-piles. At El. − 0.50 C.C.D. two 12-in. by 30-lb channels were bolted to the inside of each row of the face piling. Tie rods 2¾ in. and 2⅞ in. in diameter passed through the face piles and through these channels, tying the two rows of Z-piles or face piling together. Two 80-lb railway rails were welded to the outside of each row of the face piles, one at El. + 1.33 C.C.D., and the other at El. + 4.83. These serve as wales or guard rails. At intervals of 84 ft, diaphragms formed of steel sheet piling 34 ft long driven to a top elevation of + 4.00, across the wall, divide this wall into a series of rectangular pockets. The tops of the piles in each diaphragm are embedded in a reinforced concrete cross beam, 3 ft 6 in. wide by 2 ft 6 in. deep, at each end of which is anchored a cast steel mooring post.

This wall was filled to El. + 5.50 C.C.D. with broken rock ranging in size from ¼ in. to 20 in. Above this level was placed a 6-in. crushed-stone top finished with limestone screenings. This roadway top was given a 2-in. crown at the center.

The south basin wall was designed as a gravity dam, to sustain water pressure due to a head of 7.83 ft, from El. + 5.83 C.C.D., to El. − 2.00. The base of the wall was assumed at El. − 26.00 C.C.D. and the width at the base, 23 ft. No wave action and no surcharge was assumed. Weight of rock fill was assumed at 90 lb per cu ft "in the dry" and at 65 lb per cu ft submerged. The face piles were designed as vertical, simple beams with a cantilever at the top, to carry the horizontal load caused by the submerged rock fill and a head of 7.83 ft of water, the points of support being assumed at El. − 0.50, the elevation of the tie rods, and at El. − 30.11, the calculated first point of contraflexure below the lake bottom.

Lock Gates.—Fig. 5 is a view of the lock gates. The requirement that these gates be capable of withstanding a head of water in either direction and also remain closed against surges from the lake eliminated ordinary miter gates from consideration. A modified form of sector gate was adopted. The word "sector," as used in this paper, applies to the shape of a horizontal cross section through the prism (one sixth of a circular cylinder) forming one leaf of a set of lock gates. The sector gate, as a lock gate, had previously been developed by the engineers of The Sanitary District of Chicago and had been used as such in the controlling lock at the head of the Calumet-Sag Channel, near Blue Island, Ill. This sector gate had been developed from the idea of the sector dam used to control flow in the Main Drainage Channel at the Lockport Power House and it, in turn, had come from the idea used in the bear-trap dam at the old controlling works at Lockport, Ill. This bear-trap dam, properly counterweighted or supported by water pressure, inside or beneath the dam, was stable in any position. The same was true of the sector dam at the Lockport Power House, which was placed with its curved face upstream and, with a tight sheet-steel top, was controlled in any desired position by merely varying the pressure of water inside the dam.

The sector gates at the Blue Island lock had a tight sheet-steel curved (circular arc) face on the upstream side. All stresses were carried to the hinge, at the center of the circle, through framed members. Since these pressures all acted radially each gate leaf was stable in any position, and flow through the lock and through the Calumet-Sag Channel was controlled by setting the gates in any partly opened position. Filling and emptying valves were provided in the Blue Island lock for use when it was performing its function as a navigation lock. The U. S. Army Engineers had used the same idea in lock gates in some of the canals leading from Lake Okeechobee, in Florida. These locks were used for both navigation and flood control. The Army Engineers had gone a step further and had omitted filling and emptying valves and filled these locks by merely opening the upstream gates and emptied them by opening the downstream gates. Tests indicated that this could be done without danger to water craft in the lock at the time.

The foregoing considerations led to the adoption of sector gates for the lock at the mouth of the Chicago River and the omission of filling and emptying valves. The lock at Blue Island was 50 ft wide by 400 ft long, and the travel of each leaf of each set of the sector gates there was 25 ft. The locks in the canals in Florida were even smaller. The Chicago River lock, being 80 ft wide by 600 ft long, involved greater quantities of water for filling and a travel of 40 ft for each leaf of the sector gates. This consideration led to the adoption of triangular shaped gate leaves, giving three passages for the admission of water to the lock when the gates were in a partly opened position, in place of the one central passage possible if true sector gates had been used.

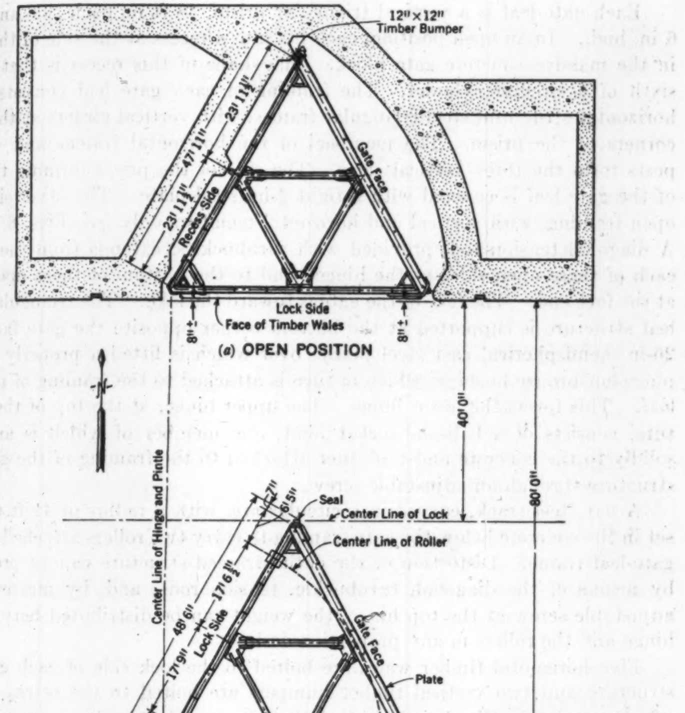

12″×12″ Timber Bumper
23′ 11½″
47′ 11″
Gate Face
23′ 11½″
Recess Side
Lock Side
Face of Timber Wales
8″±
8″±
(a) OPEN POSITION
40′ 0″
80′ 0″
Center Line of Hinge and Pintle
7″
6′ 5″
Seal
Center Line of Lock
Center Line of Roller
17′ 6½″
48′ 6″
Lock Side
Gate Face
17′ 9″
Plate
Seal
6′ 2½″
Recess Side
Center Line of Roller
47′ 11″
(b) CLOSED POSITION
47′ 11″
El +7.0
Center Line of Hinge
Web 108″ × ½″ (Bent)
6′ 5″
31′ 5¼″
Bend Line
Center Line of Hinge and Pintle
El −24.44
Center Line of Roller
El −26.44
2′ 5″
1″
(c) ELEVATION OF LOCK SIDE OF GATE

FIG. 5.—PLAN AND ELEVATION OF LOCK GATES

Each gate leaf is a vertical triangular prism, 48 ft on each side and 30 ft 6 in. high. In an open position, each fits into a recess at the side of the lock, in the massive concrete gate block. The shape of this recess is that of one sixth of a circular cylinder. The framing of each gate leaf consists of six horizontal structural steel triangular frames, with vertical girders at the three corners of the prism. The members of the horizontal frames and vertical posts form the three vertical sides. The side of the prism forming the face of the gate leaf is covered with a tight ⅜-in. steel plate. The other sides are open framing, with vertical and horizontal members only (see Figs. 6 and 7). A diagonal tension rod, provided with turnbuckles, extends from the top of each of these open sides at the hinged end to the bottom of these open sides at the face end. The face of the gate is toward the lake. The triangular gate-leaf structure is supported at the bottom corner opposite the gate face on a 26-in. hemispherical cast steel pintle, over which is fitted a properly shaped phosphor-bronze bushing, which in turn is attached to the framing of the gate leaf. This forms the lower hinge. The upper hinge, at the top of the structure, consists of a ball-and-socket joint, one member of which is anchored solidly to the concrete and the other attached to the framing of the gate-leaf structure through an adjustable screw.

A flat steel track, curved in a circular arc, with a radius of 41 ft 6 in., is set in the concrete below the gate framing to carry two rollers attached to each gate-leaf frame. Distortion of the entire framed structure can be prevented by means of the diagonal, turnbuckle, tension rods; and, by means of the adjustable screw at the top hinge, the weight can be distributed between the hinge and the rollers in any proportion desired.

Five horizontal timber wales are bolted to the lock side of each gate-leaf structure and two vertical timber bumpers are bolted to the recess side, at effective points. Checkered steel plate walks and handrails are provided on all three sides of this structure, at the top. When the gates are closed these walks afford passage from one side of the lock to the other. Rubber sealing strips are provided at the bottom and the two vertical sides of each gate-leaf face.

Operation of each gate leaf is by means of a strut, one end of which is hinged to the upper horizontal frame of the structure. Attached to one side of this strut is a rack which meshes with a pinion driven by an electric motor.

A flat steel track, 13 in. wide and 3 in. deep, curved in a circular arc with a radius of 41 ft 6 in., is set in the concrete below the gate framing to carry the rollers attached to each gate-leaf frame. Each roller is of cast steel 20 in. in diameter and 10 in. long, slightly beveled to run on the horizontal curved track. It is finished on both its rolling surface and on its inside to receive a phosphor-bronze bushing. The bushing fits around an 8-in. diameter pin of cold-rolled steel which is finished all over. The pin is keyed to a heavy structural steel frame which in turn is rigidly attached to the bottom member of the frame of the gate leaf. The bushing is provided with grease grooves which are fed from holes to the center of the pin, where a central hole extends to one end and is connected to a ¼-in. hard drawn copper tube that extends up to the

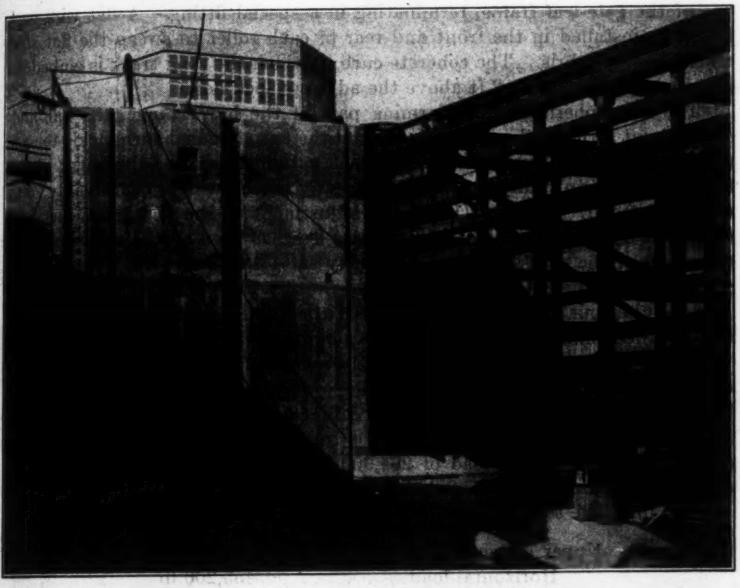


FIG. 6.—VIEW FROM SILL OF WEST GATE FOUNDATION STRUCTURE

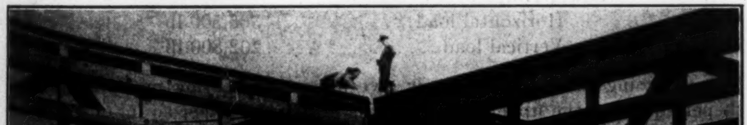


FIG. 7.—VIEW OF WEST LOCK GATE IN CLOSED POSITION DURING CONSTRUCTION

top of the gate-leaf frame, terminating in a special fitting. A flat wire push broom is installed in the front and rear of each roller to sweep the flat steel track clear of debris. The concrete curb in which the steel track is embedded is raised approximately 2 ft above the adjacent lock floor.

The hemispherical pintle forming part of the lower hinge, the ball-and-socket joint forming the upper hinge, and the two diagonal turnbuckle tension rods on the two downstream sides of each gate leaf enable adjustment of this framed structure to any position and to any extent which, conceivably, may ever be needed. To date (1940) the adjustment of the lock gates has been such that 80% of the weight of each gate leaf has been on the rollers and the operation of the gates has been extremely smooth. This lock is operated on an average of more than 50 times daily during the open navigation season. Each of the four gate leaves weighs 101 tons and is operated easily by two motors, the combined power of which amounts to only 26½ hp.

The maximum horizontal load, due to a head of 8 ft of water, is 250,900 lb on the upper hinge and 362,200 lb on the lower hinge. The maximum vertical load on a roller, due to the weight of a gate leaf, is 83,400 lb, and on the pintle is 41,800 lb. When the rollers are not functioning the upper hinge receives an additional horizontal load of 204,300 lb, and the lower hinge receives an additional horizontal load of 204,300 lb and an additional vertical load of 161,000 lb. The total maximum loads on the hinges are, therefore:

Upper Hinge	
Horizontal load	455,200 lb
Vertical load	0
Lower Hinge	
Horizontal load	566,500 lb
Vertical load	202,800 lb

The foregoing assumed that loads give maximum bearing pressures of 1,720 lb per sq in. on the upper hinge bushing and 1,410 lb per sq in. on the lower hinge bushing.

Important features of the gates in the lock at the mouth of the Chicago River are: (1) Simplicity of fabrication because of the flat gate face and the use of few built-up members in the frame work; (2) the ability to adjust the gate leaf and remove distortion; (3) the equilateral triangular shape of the gate leaves makes them self-balancing horizontally; (4) the design of the gate hinges is such that no binding can occur; (5) the operating strut and rack and pinion are all above high water; (6) and the triangular shape of the gate leaf, with the sector shape of its recess, permits water to flow around both ends of the gate face when water levels are being equalized.

Concrete Gate Blocks.—The concrete gate blocks at each end of the lock were built "in the dry," inside of cellular type cofferdams (see Fig. 8). The foundation of each of these structures consists of a heavily reinforced concrete floor, supported on wooden piles. The dimensions of this floor slab are 110 ft longitudinal with the axis of the lock by 196 ft 6 in. transverse to the lock, with a thickness of 7 ft, except for the central 40 ft and for the part under the gate recesses where the thickness is reduced to 4 ft. Each of these heavy floor slabs is supported on 1,632 wooden piles, 27 ft long under the 7 ft thick section and 30 ft long under the 4 ft thick section, driven to El. − 59.00 C.C.D. on a maximum of 4-ft centers. The piles project up about 9 in. into the concrete floor. Steel sheet piles extending up 1 ft into the concrete slab, 1 ft 6 in. from the outer edges, completely surround the wooden piles and prevent the soil from squeezing out. These sheet piles were driven to El. − 45.00 C.C.D. This concrete floor slab was reinforced principally as a beam, spanning the soil across the lock. The top of this floor was made at El. − 26.44 C.C.D.

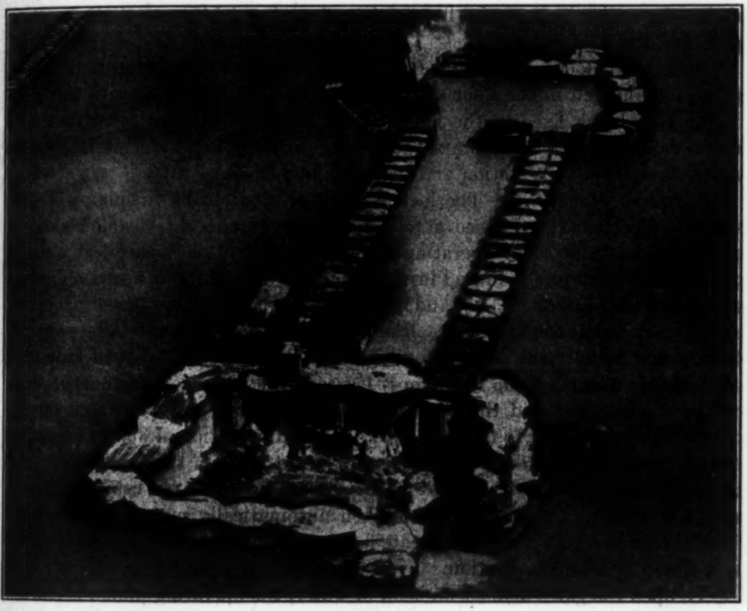


FIG. 8.—GENERAL VIEW OF WALL CONSTRUCTION AND COFFERDAM FOR CONSTRUCTION OF CONCRETE GATE BLOCK "IN THE DRY"

On each side of the lock a concrete gate block was constructed, 110 ft long and 53 ft 9 in. wide from El. + 7.00 C.C.D., down to the floor slab at El. − 26.44. A 60° sector shaped gate-leaf recess, with a radius of 52 ft 6 in., was formed in each of these gate blocks. On the floor slab between the gate blocks a V-shaped curb, 2 ft high and 4 ft wide, was constructed as a seal for the lock gates when in a closed position. Two curved curbs were also constructed, 1 ft high and 2 ft 3 in. wide, one upstream and one downstream from each lock gate, to serve as bottom guides and supports for cofferdams in case it should be desirable to dewater the space around the gates for repairs. A similar straight curb was constructed along the outer edge of each gate-leaf recess to serve as a guide and support for a straight cofferdam in case it should be necessary to dewater a gate recess. Two curved track foundations were also built on the floor slab, embedding heavy track plates at El. − 24.52 C.C.D., on which the rollers of the gate leaves operate.

Each gate foundation structure was designed in various ways: (*a*) As a rigid frame, with the entire structure "in the dry" and the gate leaves in their recesses (a condition occurring only during construction); (*b*) with the entire structure submerged in water to El. + 4.00 C.C.D., and with wave force of 6,000 lb per lin ft applied to the side of one gate block at El. + 4.00; and (*c*) with the two curved temporary cofferdams in place, the gate space across the lock, dry, with surrounding water at El. + 4.00, full hydrostatic uplift under the entire base of the floor slab and no bond between the supporting piles and the concrete slab.

Each gate block was designed as a rigid frame: (*a*) With the walls of the gate block hinged in the floor slab at El. − 28.44 C.C.D.; (*b*) with the walls of the gate block fixed in the floor slab at El. − 28.44; and (*c*) with a gate recess cofferdammed and pumped dry, with surrounding water at El. + 4.00 and a force from wave action and boats applied at El. + 4.00.

Operating Machinery.—The lock is filled or emptied by opening one gate slowly. Fig. 9 shows the east gate being opened during a test under a differential head of 8 ft. The operating machinery for each lock gate leaf consists of two electric motors mounted in conjunction with a suitable speed reduction unit. A small motor (1.5 hp and 1,145 rpm) is used for low-speed operation at the start and at the finish of each movement of the gate leaf. A large motor (25 hp and 1,160 rpm) is used for high-speed operation. A speed reduction of 30,000 to 1 is made (in four steps) between the pinion on the output shaft of the small motor and the operating pinion meshing with the rack on the operating strut hinged to the gate leaf. A speed reduction of 350 to 1 is made (in two steps) between the pinion on the output shaft of the large motor and the operating pinion. The large motor can be cut in or out while the small motor is still operating. A limit switch automatically cuts off the current from the smaller motor when the gate leaf has reached its limit of travel either in a closed or an open position.

To open the gates against no head of water, the operating pinion is started at high speed (both motors running) and operated at this speed until the gate leaf is within a few inches of its fully open position, when the large motor is cut out automatically, the small motor continuing operation until the gate leaf is in its fully open position, when the limit switch cuts out the small motor. In the operation of closing the gates the procedure is almost identical; the operating pinion runs at high speed until the gate leaf is within a few inches of its closed position, when the large motor cuts out, the gate being moved on, at slow speed by the small motor, to its fully closed position, when the small motor is stopped. The time required for this operation is about 3 min, during the first 15 sec of which both motors move the gate leaf 41 in. (2.7 in. per sec) and bring the movement to full speed. During the next 105 sec (1¾ min) both motors move the gate leaf 565 in. (5.4 in. per sec) and during the final 60 sec the small motor moves the gate leaf 4 in. into its final position at an average speed of ⅔ in. per sec.

To open the gates against a head, there is one additional step; namely, the operating pinion is run at low speed by the small motor for a predetermined length of time depending upon the existing difference in head, until water levels on both sides of the gate are equalized. When this has been accomplished, the large motor is automatically cut in, the movement accelerated, and the gate leaf moved into its final position in about three additional minutes, against no head. To open the gates against a 6-ft head, a period of 480 sec (8 min) in which the gate leaf travels 32 in. (0.067 in. per sec) is required for equalizing the water levels. The total time for operation against a 6-ft head thus becomes approximately 11 min. The time needed for operation against a head of 2 ft, which may be the future normal condition, is about 9 min.

FIG. 9.—VIEW OF EAST GATE BEING OPENED DURING A TEST UNDER 8 FT OF DIFFERENTIAL HEAD

The operating machinery of each gate leaf is designed to deliver a normal force of 24,000 lb to the recess side of this framed structure at the top horizontal frame and at a distance 36 ft from the vertical center line of the hinges. The maximum resistance to movement of a gate leaf is estimated at 23,700 lb, as follows:

Description	Resistance, in pounds
From current of 4 ft per sec	8,050
Movement of leaf against still water	130
Surface friction of leaf	10
Friction on pintle, from vertical load	130
Collar-bearing friction on rollers	560
Axle friction of rollers	11,100
Rolling friction of rollers	3,720
Total	23,700

Arrangements have been made for the attachment of cables to the gate leaves, near the bottom, for emergency operation. Hand-operating machinery has also been provided, with a speed reduction of 280 to 1 from the hand crank to the operating pinion. Four men turning the hand crank at a rate of 30 rpm will open or close a gate leaf in about an hour.

Operators' Houses.—An operator's house, or lock control house, has been constructed on each of the four concrete gate blocks to house the control equipment for operating the lock gates and for lighting and signaling. These houses are of reinforced concrete construction, with flat concrete roofs, or decks. The main operator's house, on the northwest gate block, is 20 ft wide, 24 ft long, and 12 ft 9 in. high, outside dimensions. The other three houses are 11 ft wide, 16 ft long, and 12 ft 9 in. high, above El. + 7.00 C.C.D. All houses have double sash windows on all four sides. Steel ladders, anchored to the walls of the houses, give outside access to the roofs, or decks, which are enclosed by pipe handrailing.

Each lock-control house contains a differential gage showing the difference in water levels between the lock and lake and between the lock and river. The main control house contains a recording gage which makes a permanent record of the water levels in the lake and in the river. Each control house is equipped with an oil-burning heater, a telephone or telephone extension, and houses a control stand and electric switchboard.

Electrical Features.—Electric energy for operation of the lock machinery, for lighting, etc., is received from the local public service company at 12,000 volts at the main lock control house (northwest operator's house), where two 12,000–480 volt power transformers and two 12,000–230–115 volt lighting transformers are used to reduce it to the proper voltage. It is carried to various outlets, for use, through conduits laid in the concrete slab, or grid, at the top of the lock.

Power lines serve a 1.5-hp and a 25-hp motor in each of the four lock control houses. Both east gate leaves can be operated simultaneously or independently from either of the easterly control houses and both west gate leaves can be operated from either of the westerly control houses. There is no interconnection between the east and the west gates.

The lighting system for the lock, guide walls, and north basin wall is a single-phase, 3-wire, 230–115 volt, system, with grounded neutral. Lighting standards (27 ft high, with a horizontal arm supporting a sodium-vapor type lamp, 23 ft above the top of the walls) are placed on each of the four gate blocks, and along the walls at 140-ft intervals.

A complete signal system has been installed, with red and green stop-and-go lights at each end of the lock. There are also red and white flashers to mark the ends of the guide walls. The flashers are gas-operated. A motor-driven air compressor with storage tank in the main control house supplies an air whistle that can be operated electrically from any one of the four lock control houses.

The telephones in the four control houses are equipped with loud gongs, and are interconnected for communication between the control houses and for outside calls at all houses.

Lock Floor.—The only part of the lock having a solid floor is that between the concrete gate blocks at the ends of the lock. The bottom of the lock proper is paved with precast reinforced concrete floor slabs, 8 in. thick. These slabs are 6 ft wide by 27 ft long. They were placed under water by means of derricks. The services of divers were required to fit the slabs closely together, this being the only operation in the construction of the entire project requiring the use of divers. The bottom of the lock was dredged and swept to proper grade and as nearly level as possible with dredging equipment. When the concrete slabs had been placed and fitted, they were leveled by placing a weight on any corner that extended above floor level and by removing the weight as soon as this part of the slab had been pressed down sufficiently into the comparatively soft clay.

The floor of precast concrete slabs is 81 ft wide and fails to cover that part of the lock bottom in the angle formed by the curved walls of the sheet pile cells. This part of the lock, back of the face of the lock walls, has no floor other than the original clay. It is not believed that this clay will scour because it is at the bottom corners of the lock where water currents set up by boat propellers will scarcely penetrate; or, if they do penetrate, they will be so diminished as to have no scouring intensity. The lock floor as originally planned was to have been formed of precast slabs 6 ft by 6 ft by 8 in. thick and the change to slabs 6 ft by 27 ft was at the request of the contractor to lessen the number of joints and labor of fitting.

Other types of lock floors were considered seriously by the designers, preferably a monolithic concrete floor placed "in the dry" and a monolithic floor placed "in the wet" by the tremie method. Any monolithic floor would have covered the entire bottom of the lock. The placing of the floor "in the dry" would have necessitated the complete dewatering of the lock; but this was not permitted although the lock walls were designed against such a contingency and it would have been an excellent test of the stability and watertightness of this type of wall. In placing the precast floor slabs, the water level inside the lock was lowered approximately 8 ft, giving a depth of water of about 16 ft in which to do this work.

A suggestion of omitting the lock floor, another of making a rubble floor of one man stone, were made but not seriously considered for the reason that propellers of large boats or tugs would create water currents at the ends of the lock which would scour the clay or move the stones possibly into positions which would interfere with the movement of the lock gates.

The floor of the lock at the ends, between the concrete gate blocks, is at a level 1 ft below the main floor and 2 ft below the top of the V-shaped curb on which the lock gates close, thus forming shallow debris pits which are expected to catch any heavy debris that might otherwise interfere with the operation of the lock gates.

Crest Wall.—A continuous reinforced concrete crest wall, 4 ft high, 2 ft 3 in. wide at the base, and 2 ft 7 in. wide at the top, with the windward or southerly face curved to throw back waves striking it, was constructed along the top of the southeast guide wall, the south lock wall, and the southwest guide wall. The northerly or lee face of this wall is vertical, paneled for architectural effect, and located on a line 9 ft 6 in. from the lake side of the lock and guide walls. There is a clear reach of almost 1.5 miles in Chicago Harbor to the south and southeast of the lock, from which considerable waves can be produced. The northerly side of the lock is protected by the Navy Pier and an existing breakwater, both of which extend out in Chicago Harbor much farther than the lock and its guide walls.

Control Gates and Flood Gates.—Sections of wall 59 ft long in the north basin wall and in the south basin wall are constructed of concrete, supported on wooden piles, forming gate blocks in which the control gates and flood gates are installed. Four control gates and four flood gates are installed in each location. At each set of gates (control and flood) there is a passage 10 ft by 10 ft, clear opening, through the concrete wall, the top being at El. − 8.00 C.C.D., and the bottom at El. − 18.00. This should bring the top of the opening always well below the ice and debris line. On the lake side of the wall this opening is flared for a distance of 5 ft, being 12 ft wide and 13 ft high at the lake face of the wall and 10 ft square at a point 5 ft inward. The 10 ft square cross section is continued for a distance of 9 ft, where it terminates in a flume with a width of 11 ft. The bottom of the flume slopes down to El. − 19.00 C.C.D., at the inner face of the basin wall and the top slab of the basin wall, 18 in. thick, forms a roof over the flume, well above the normal water surface.

A sluice gate 10 ft 5½ in. wide by 10 ft 3 in. high is set vertically 7 ft inside the lake face of the basin wall, or 2 ft inside the outer end of the square shaped opening (see Fig. 10). Such a sluice gate can be set at any desired opening and used to control the flow of water from the lake into the river.

At the inner end of the square shaped opening, a flap gate, 10 ft 8 in. wide and 10 ft 9½ in. high, hinged at the top, is installed in the flume to serve as a flood gate. This flap gate is made of 8-in. by 12-in. horizontal pieces of dense Douglas fir timber, joined horizontally with 1-in. by 2-in. white oak splines and reinforced vertically with three ⅝-in. by 8-in. steel plates on the lake side and by three pairs of 5-in. by 3½-in. angles on the river side of the gate. The gate support consists of a 3½-in. steel pipe over which rest two steel hooks attached to the upper edge of the gate. The flap gate, when in a closed position, bears against the faces of the 10-ft by 10-ft opening through the concrete gate block.

Steel plate counterweights are provided at the bottom of each flap gate, to overcome buoyancy, and thus the gate is made slightly heavier than the water it displaces. When water flows from the lake through a sluice gate, its corresponding flap gate will swing open at a vertical angle, the magnitude of which will depend upon the existing current of the water. If the flow should reverse toward the lake, the flap gate will close automatically and stop all such flow. The flap gates thus serve as automatic flood gates and are "fool proof," in that no reliance is placed upon human judgment as to when they should be closed. This feature is particularly important in this location, where the lake level frequently changes as much as 1 ft in an hour, due to a sudden shift in the direc-

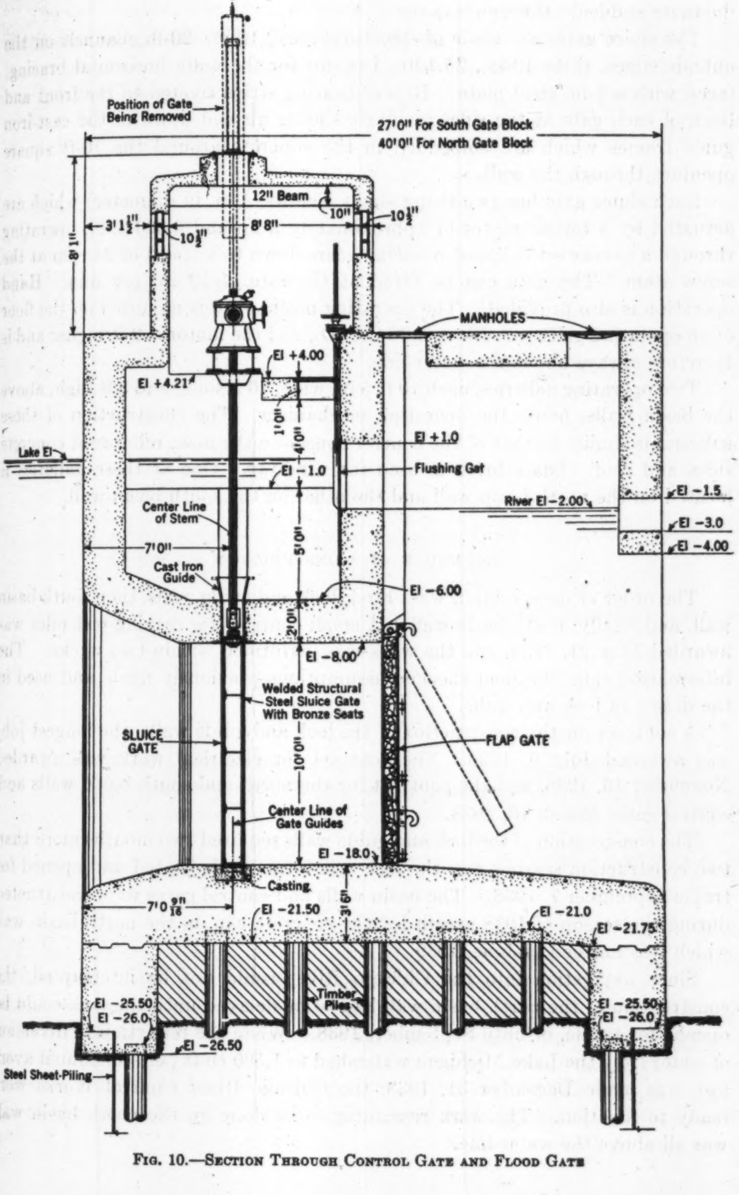

Position of Gate Being Removed
27'0" For South Gate Block
40'0" For North Gate Block
12" Beam
10"
10½"
3'1½"
8'8"
10½"
8'1"
MANHOLES
El +4.00
El +4.21
1'0"
4'0"
El +1.0
Lake El
El −1.0
Flushing Gat
El −1.5
River El −2.00
El −3.0
El −4.00
Center Line of Stem
7'0"
5'0"
Cast Iron Guide
El −6.00
2'0"
El −8.00
Welded Structural Steel Sluice Gate With Bronze Seats
SLUICE GATE
FLAP GATE
10'0"
Center Line of Gate Guides
El −18.0
Casting
7'0 9/16"
El −21.50
3'0"
El −21.0
El −21.75
Timber Piles
El −25.50
El −26.0
El −25.50
El −26.0
El −26.50
Steel Sheet-Piling

FIG. 10.—SECTION THROUGH CONTROL GATE AND FLOOD GATE

tion of wind, and where "seiches" occasionally occur, in which lake levels fluctuate suddenly through a range of 4 to 6 ft.

The sluice gates are made of structural steel, 10-in., 20-lb channels on the outside edges, three 10-in., 25.4-lb., I-beams for the main horizontal bracing, faced with a ⅜-in. steel plate. Bronze bearing strips riveted to the front and back of each gate at the sides facilitate sliding up and down in the cast-iron guide frames which are embedded in the concrete around the 10-ft square openings through the wall.

Each sluice gate has two rising stems (screws, 4 in. in diameter) which are actuated by a torque motor of approximately 5 hp and 3,600 rpm, operating through a proper set of speed-reducing gears down to a speed of 24 rpm at the screw stem. The gate can be lifted at the rate of 12 in. per min. Hand operation is also provided. The operating mechanism is mounted on the floor of an operating gallery at El. + 4.00 C.C.D., and the motor is 2 ft higher and is therefore always above the water line.

Two operating galleries, each 10 ft 5 in. wide, 70 ft long, and 8 ft high, above the basin walls, house the operating mechanism. The construction of these galleries is similar to that of the control houses on the lock, reinforced concrete sides and roof. Glass brick is used for windows. One of these galleries is located on the north basin wall and the other on the south basin wall.

SCHEDULE OF CONSTRUCTION

The order of construction was: First, lock and guide walls, then south basin wall, and finally, north basin wall. A small contract for driving test piles was awarded May 21, 1936, and the work was performed within two weeks. The information thus obtained checked assumptions previously made and used in the design of lock and walls.

A contract for the construction of the lock and guide walls, the longest job, was awarded July 9, 1936. The contract for electrical work was awarded November 16, 1936, and the contract for the north and south basin walls and control gates March 16, 1938.

The construction of the lock and guide walls required two months more than two construction seasons and the lock was formally dedicated and opened for traffic September 7, 1938. The basin walls and control gates were constructed during the season of 1938 except for the concrete cap on the north basin wall which was finished in June, 1939.

Since navigation into the Chicago River could not be interrupted, the construction of the north basin wall had to be delayed until the lock could be opened for traffic, or until September, 1938. When the reduction in diversion of water from the Lake Michigan watershed to 1,500 cu ft per sec, annual average, was made December 31, 1938, the Chicago River Control Works were ready to function. The work remaining to be done on the north basin wall was all above the water line.

COSTS

The approximate cost of the Chicago River Control Works was $2,704,000, distributed among the various items, as follows:

Test piles		$ 4,600
Lock:		
Cofferdams	$105,000	
Gate structures	554,000	
Gates	177,000	
Walls	467,000	
Floor	42,000	
Control houses	23,000	
Fender piles	5,000	
		1,373,000
Guide walls		579,000
South basin wall		223,000
North basin wall		53,000
Control Works:		
Gate blocks	114,000	
Gates	32,000	
Operating galleries	6,000	
		152,000
Electrical work		73,700
Miscellaneous work		14,100
Field office		7,000
		2,479,400
Government supervision (1.05%)		26,000
Engineering expense—design, supervision, inspection, testing (8.01%)		198,600
Total cost		$2,704,000

OPERATION OF LOCK

In operating the lock gates the first step is to equalize the water surfaces on the two sides of the gate by separating the gate leaves slowly and letting the water flow between them as well as through the two openings at the sides of the lock between the faces of each gate leaf and the edge of the concrete wall of the gate recess. With the operating machinery as now installed, the gate leaves separate at the center at the rate of 4 in. per min. The combined openings at the sides of the lock are approximately 15% greater than the center opening; that is, when the outer edge of a gate leaf has traveled 6 in. from the center line of the lock, its other edge has moved back 6 in. into the gate recess and has left an opening about 7 in. wide between the concrete edge of this recess and the face plate of the gate. When the gate leaves are separated 12 in. at the center of the lock, the combined width of the side openings is about 14 in. Essential flow areas, clearances, and data are shown in Figs. 11 and 12.

Formulas have been developed for calculating the time required to fill or empty the lock for a given difference in head. The formulas have been based on the assumption that the total widening of the three openings past a set of gates is at a uniform rate of 9 in. per min. The formula used for determining the discharge, in cubic feet per second, for any given condition is a combination of the Francis weir formula,

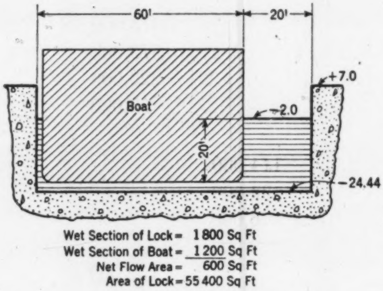

60'
20'
Boat
+7.0
-2.0
20'
-24.44
Wet Section of Lock = 1 800 Sq Ft
Wet Section of Boat = 1 200 Sq Ft
Net Flow Area = 600 Sq Ft
Area of Lock = 55 400 Sq Ft

FIG. 11.—TYPICAL CROSS SECTION OF LOCK

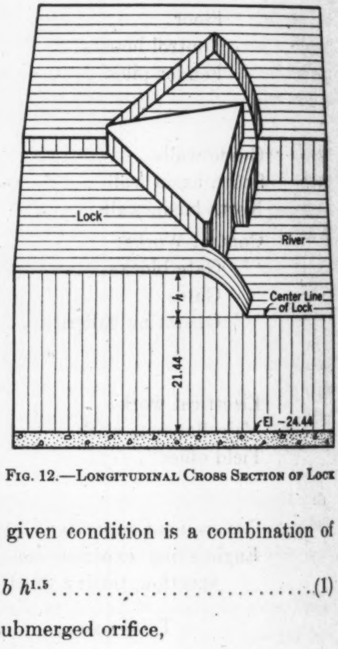

Lock
River
h
Center Line of Lock
21.44
El -24.44

FIG. 12.—LONGITUDINAL CROSS SECTION OF LOCK

$$Q = 3.33\, b\, h^{1.5} \qquad (1)$$

and the formula for discharge through a submerged orifice,

$$Q = A\, (2\, g\, h)^{0.5} \qquad (2)$$

to both of which are applied a contraction coefficient of 0.95.

The formula for rate of filling the lock is:

$$Q = 7.62\, b\, d\, h^{0.5} - 4.45\, b\, h^{1.5} \qquad (3a)$$

The formula for emptying the lock is:

$$Q = 7.62\, b\, d\, h^{0.5} + 3.17\, b\, h^{1.5} \qquad (3b)$$

in which Q = discharge, in cubic feet per second; b = total width, in feet, of three orifices; d = depth of water, in feet, in orifices at end of filling or emptying operation; and h = head, in feet, between water inside lock and water outside lock, at any instant.

The corresponding formulas for time, in seconds, are as follows—for filling lock:

$$T = \frac{1{,}872}{d^{0.25}} \left[\log \frac{1.31\, d^{0.5} + h_1^{0.5}}{1.31\, d^{0.5} - h_1^{0.5}} - \log \frac{1.31\, d^{0.5} + h^{0.5}}{1.31\, d^{0.5} - h^{0.5}} \right]^{0.5} \ldots\ldots (4a)$$

for emptying the lock:

$$T = \frac{1{,}900}{d^{0.25}} \left[\tan^{-1} \left(\frac{h_1}{2.40\, d} \right)^{0.5} - \tan^{-1} \left(\frac{h}{2.40\, d} \right)^{0.5} \right]^{0.5} \ldots\ldots (4b)$$

in which h_1 is the head, in feet, when gates start opening (a constant).

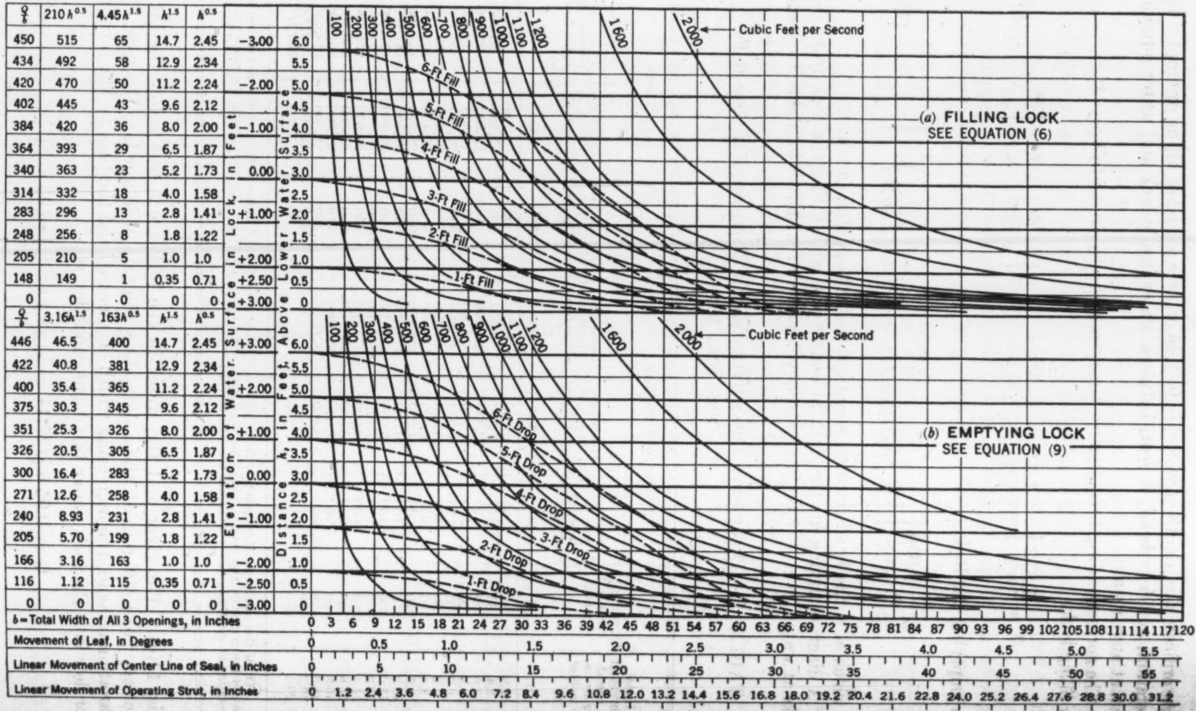
A diagram giving all the data in connection with filling the lock from the lake at water level + 3.00 has been developed and is shown in Table 1(*a*) and Fig. 13(*a*). The river and the water in the lock are h feet below the lake. For any head differential, these data show the rate of inflow into the lock, the variation in head, the increasing width of opening, and the time. The formula for flow into the lock under this head condition is

$$Q = 0.95\, b \left[(27.44 - h)\, (2\, g\, h)^{0.5} + 3.33\, h^{1.5} \right] \ldots\ldots\ldots\ldots (5)$$

TABLE 1.—DISCHARGE QUANTITIES FOR A RISE OR FALL OF 6 FT (SEE FIG. 13)

Linear movement of operating struts, in inches[a]	Cumulative time, in seconds[b]	(*a*) A 6-FT FILL			(*b*) A 6-FT DROP		
		Average rate of flow, in each 30 sec[c]	Discharge in 30 sec, in cubic feet	Total discharge, in cubic feet	Average rate of flow, in each 30 sec[c]	Discharge in 30 sec, in cubic feet	Total discharge, in cubic feet
(1)	(2)	(3)	(4)	(5)	(3)	(4)	(5)
1.2	30	90	2,700	2,700	90	2,700	2,700
2.4	60	240	7,200	9,900	250	7,500	10,200
3.6	90	410	12,300	22,200	410	12,300	22,500
4.8	120	570	17,100	39,300	550	16,500	39,000
6.0	150	710	21,300	60,600	680	20,400	59,400
7.2	180	850	25,500	86,100	780	23,400	82,800
8.4	210	960	28,800	114,900	880	26,400	109,200
9.6	240	1,040	31,200	146,100	940	28,200	137,400
10.8	270	1,100	33,000	179,100	990	29,700	167,100
12.0	300	1,120	33,600	212,700	1,010	30,300	197,400
13.2	330	1,080	32,400	245,100	1,000	30,000	227,400
14.4	360	1,000	30,000	275,100	930	27,900	255,300
15.6	390	850	25,500	300,600	850	25,500	280,800
16.8	420	600	18,000	318,600	710	21,300	302,100
18.0	450	300	9,000	327,600	560	16,800	318,900
19.2	480	....	....	....	400	12,000	330,900
20.4	510	....	....	....	40	1,200	332,100

[a] Or of the pitch circle (diameter, 20.37 in.) on the main pinion. [b] Time influences these data on the basis of the orifice on the center line of the lock widening at the rate of 4 in. per min, with the main pinion operating at 0.0375 rpm. [c] In cubic feet per second.



$\frac{Q}{b}$	$210h^{0.5}$	$4.45h^{1.5}$	$h^{1.5}$	$h^{0.5}$	Elevation of Water Surface in Lock, in Feet	Distance h, in Feet, Above Lower Water Surface
450	515	65	14.7	2.45	−3.00	6.0
434	492	58	12.9	2.34		5.5
420	470	50	11.2	2.24	−2.00	5.0
402	445	43	9.6	2.12		4.5
384	420	36	8.0	2.00	−1.00	4.0
364	393	29	6.5	1.87		3.5
340	363	23	5.2	1.73	0.00	3.0
314	332	18	4.0	1.58		2.5
283	296	13	2.8	1.41	+1.00	2.0
248	256	8	1.8	1.22		1.5
205	210	5	1.0	1.0	+2.00	1.0
148	149	1	0.35	0.71	+2.50	0.5
0	0	0	0	0	+3.00	0

$\frac{Q}{b}$	$3.16h^{1.5}$	$163h^{0.5}$	$h^{1.5}$	$h^{0.5}$	Elevation of Water Surface in Lock, in Feet	Distance h, in Feet, Above Lower Water Surface
446	46.5	400	14.7	2.45	+3.00	6.0
422	40.8	381	12.9	2.34		5.5
400	35.4	365	11.2	2.24	+2.00	5.0
375	30.3	345	9.6	2.12		4.5
351	25.3	326	8.0	2.00	+1.00	4.0
326	20.5	305	6.5	1.87		3.5
300	16.4	283	5.2	1.73	0.00	3.0
271	12.6	258	4.0	1.58		2.5
240	8.93	231	2.8	1.41	−1.00	2.0
205	5.70	199	1.8	1.22		1.5
166	3.16	163	1.0	1.0	−2.00	1.0
116	1.12	115	0.35	0.71	−2.50	0.5
0	0	0	0	0	−3.00	0

FIG. 13.—FILLING AND EMPTYING DATA; CHICAGO RIVER CONTROL WORKS

in which: h = head differential, in feet; 0.95 = orifice coefficient; 27.44 = depth of lake, in feet, above the bottom of the lock; and b = total width, in feet, of orifice.

Simplified, Eq. 5 is

$$Q = 210\, b\, h^{0.5} - 4.45\, b\, h^{1.5} \ldots\ldots\ldots\ldots (6)$$

The derivation of the equation for filling the lock from the lake at El. + 3.00 with the total of the three openings of the gates widening at a constant rate of 9 in. per min is as follows: $dD = \frac{Q}{A}\, dt = \left(\frac{210\, b\, h^{0.5} - 4.45\, b\, h^{1.5}}{55{,}400}\right) dt$ $= \left[\frac{210\,(h_1 - D)^{0.5} - 4.45\,(h_1 - D)^{1.5}}{55{,}400} \times \frac{t}{80}\right] dt$, or:

$$t\, dt = \left[\frac{996{,}000}{47.2\,(h_1 - D)^{0.5} - (h_1 - D)^{1.5}}\right] dD \ldots\ldots\ldots (7a)$$

in which: h_1 = head in feet when gates start opening (a constant); h = head in feet at any time during opening; D = total drop in head (in feet) at any time during opening = $h_1 - h$; t = time in seconds for head to drop from h_1 to h; Q = discharge in cubic feet per second = $210\, b\, h^{0.5} - 4.45\, b\, h^{1.5}$; b = total width of openings in feet = $t \div 80$; and A = area of lock surface = 55,400 sq ft. Let $u = h_1 - D$. Then $du = -\,dD$; $dD = -\,du$; and

$$t\, dt = \left(\frac{-\,996{,}000}{47.2\, u^{0.5} - u^{1.5}}\right) du \ldots\ldots\ldots\ldots (7b)$$

Let $v = u^{0.5} = (h_1 - D)^{0.5}$. Then $dv = \frac{1}{2}\, u^{-0.5}\, du$; $du = 2\, u^{0.5}$; $dv = 2\, v\, dv$; and

$$t\, dt = \left(\frac{-\,996{,}000 \times 2\, v}{47.2\, v - v^3}\right) dv = \left(\frac{-\,1{,}992{,}000}{47.2 - v^2}\right) dv \ldots\ldots (7c)$$

Integrating and simplifying: $t^2 = \left[-\,672{,}000 \times \log \frac{6.86 + (h_1 - D)^{0.5}}{6.86 - (h_1 - D)^{0.5}}\right] + C.$

When $D = 0$, $t^2 = 0$. Then, $C = +\,672{,}000 \times \log \frac{6.86 + h_1^{0.5}}{6.86 - h_1^{0.5}}$; and t^2 $= 672{,}000 \left[\log \frac{6.86 + h_1^{0.5}}{6.86 - h_1^{0.5}} - \log \frac{6.86 + (h_1 - D)^{0.5}}{6.86 - (h_1 - D)^{0.5}}\right]$; but $h_1 - D = h$; and, therefore,

$$t = 820 \left[\log \frac{6.86 + h_1^{0.5}}{6.86 - h_1^{0.5}} - \log \frac{6.86 + h^{0.5}}{6.86 - h^{0.5}}\right]^{0.5} \ldots\ldots\ldots (8)$$

The data for the filling diagram were developed with the aid of Eq. 6 for flow and Eq. 8 for time of filling.

Table 1(*b*) and Fig. 13(*b*) contain the data in connection with emptying the lock into the river at water level − 3.00 (see Fig. 12). The formula for flow

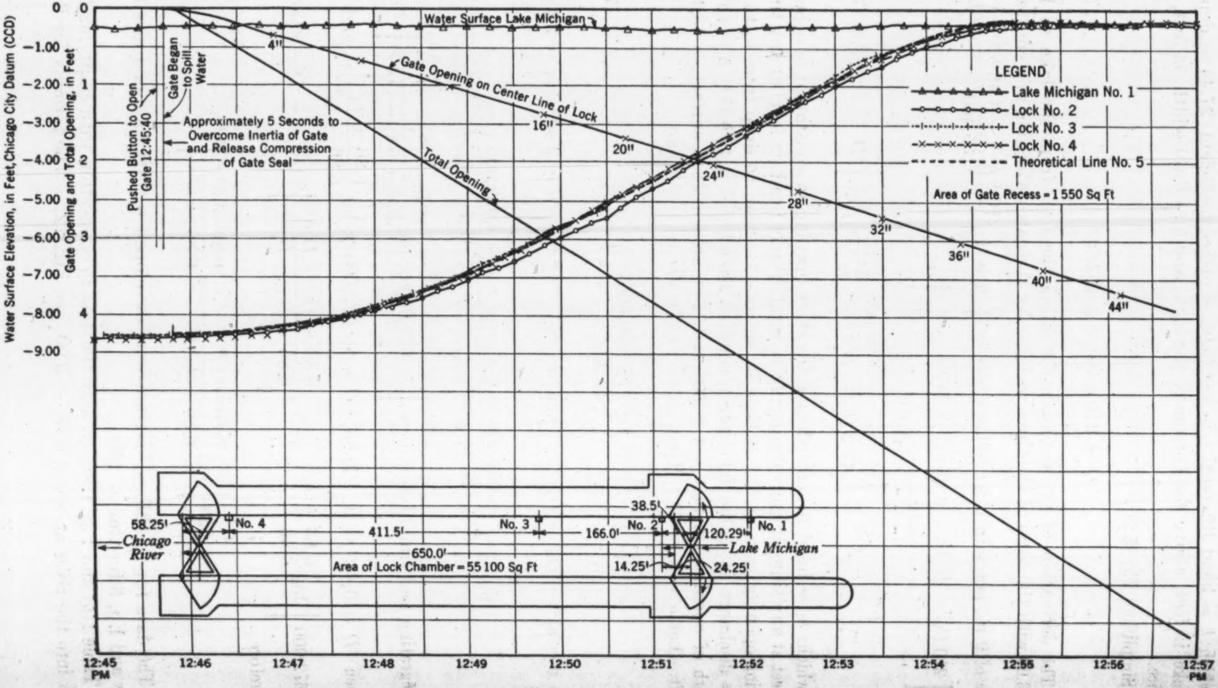


FIG. 14.—TIME REQUIRED TO FILL LOCK CHAMBER, OPERATING GATES AT SLOW SPEED; JUNE 25, 1938

from the lock under this head condition is

$$Q = 0.95\, b\, [21.44\, (2\, g\, h)^{0.5} + 3.33\, h^{1.5}] = 163\, b\, h^{0.5} + 3.17\, b\, h^{1.5} \ldots (9)$$

The derivation of the equation for emptying the lock into the river at El. − 3.00 with the total of the three openings of the gates widening at a constant rate of 9 in. per min is as follows:

$$dD = \frac{Q}{A}\, dt = \left(\frac{163\, b\, h^{0.5} + 3.17\, b\, h^{1.5}}{55{,}400} \right) dt \ldots\ldots\ldots\ldots (10)$$

Following the same procedure as in Eq. 7,

$$t = 885 \left[\tan^{-1} \left(\frac{h_1}{51.4} \right)^{0.5} - \tan^{-1} \left(\frac{h}{51.4} \right)^{0.5} \right]^{0.5} \ldots\ldots\ldots\ldots (11)$$

The data for Fig. 13(*b*) were developed with the aid of Eq. 10 for flow and Eq. 11 for time of filling. The formulas for filling and emptying the lock under the conditions of lake level at + 3.00, and river level at − 3.00, and the curves in Fig. 13 were prepared as a basis for study of the design and operation of the lock. However, for the normal range of lake levels to be expected, and for all practical purposes, the filling times for various starting head differentials can be read from these curves with very slight percentage of error.

An actual test of time of filling the lock was made June 5, 1938, with the water level at El. − 0.50 C.C.D. outside the lock and at El. − 8.60 inside the lock at the beginning of the test. The results of this test checked within 1% of the theoretical results computed from the formula for time of filling the lock. Fig. 14 contains curves of the water levels at three points inside the lock and of the lake outside the lock during the test. The heavy dash line is the theoretical line, plotted from data derived from the general formulas for flow and for time of filling.

In equalizing the water surfaces on the two sides of the gates by permitting the water to flow through orifices between and around the gates there are always two variables. The combined width of the three openings is increasing at a practically constant rate, the head is lessening at a varying rate and the depth of the openings is lessening at a rate similar to the head, but of comparatively small effect considering the total depth. The net result is a decreasing head and an increasing area of opening. The flow at first is at maximum velocity because of maximum head, but of small volume because of minimum area. The volume of flow increases to its maximum when about 60% of the width of the flow opening has been attained, the head then being about 40% of the maximum or initial head. The volume of flow then decreases (more sharply than it had increased) until near the end of the time of equalization the area of the orifices is maximum and the velocity is minimum.

It is important that the volume of inflow to the lock should be kept low in order that no currents should be set up inside the lock which might be disturbing to water craft in the lock at the time. In the test of June, 1938, with a maximum initial head of 8 ft—the worst possible condition—the maximum rate of inflow was about 1,300 cu ft per sec. Approximately 600 cu ft per sec of this came through the central opening and about 350 cu ft per sec from each of the side openings. The velocity of water longitudinally down the lock at the influent end could not exceed 0.65 ft per sec, with a flow of 1,300 cu ft per sec and a cross section of 2,000 sq ft. At the opposite end, of course, the current was zero. Even with a large boat in the lock, reducing its cross section, no current could be created dangerous to craft inside the lock.

Benefits from Control Works

The only disadvantage caused by the control works at the mouth of the Chicago River will apply to navigation. These works necessitate the locking of every boat entering the Chicago River from Lake Michigan or passing from the river to the lake, entailing a delay of approximately 15 min for each operation. This penalizes only a small percentage of the lake boats using Chicago Harbor since most of the larger boats dock at the Navy Pier outside the control works. The facilities at this pier are ample.

The advantages of the control works are the creation of an inner harbor, the better control over flow in the Chicago River and Main Drainage Canal and, most important, the insurance against the reversal of flow of the Chicago River into Lake Michigan.

The inner harbor, about 2,000 ft long from north to south and about 1,700 ft wide from east to west, comprises about 17 acres of protected water, safe for river barges. This harbor is now used for the interchange of cargoes between lake boats and river barges and a future development will undoubtedly be the interchange of such freight across the north pier, widened and provided with warehouses and unloading and conveying equipment.

The regulation of flow in the river and canal system in the past has been solely by the manipulation of the controlling works at Lockport, 36 miles inland from the mouth of the Chicago River. Close control over flow in the Chicago River has been difficult, at times, because of sudden fluctuations of Lake Michigan and because of inflow from rainfall runoff which could occur in the Chicago River in less time than that required to change flow conditions by operating the Lockport Controlling Works. The level of Lake Michigan frequently changes as much as 1 ft in one hour due to a shift in wind. Runoff from rains can enter the Chicago River in volume within three hours after rain has started to fall. It can enter the Main Chicago River within less than one hour, from the "loop" area. Three hours will elapse after a change in flow conditions is made at Lockport before the first effect of this change can be noticed in the Main Chicago River and from six to nine hours are required to produce a substantial change in flow conditions near the mouth of this river. For this reason, in the past, it was necessary to anticipate rainfall by three to six hours and establish flow conditions which would permit the inflow from storms without danger of reversal of the river.

When the average flow in the canal was around 10,000 cu ft per sec it was necessary to increase the flow only two or three times per year to handle the worst storms. The maximum runoff from rains is now about 12,000 cu ft per sec. This will increase in the future, as paved areas in Chicago increase. When the average flow in the canal was around 6,600 cu ft per sec (diversion from Lake Michigan watershed 5,000 cu ft per sec, annual average, in addition to 1,600 cu ft per sec sewage flow) it was necessary to increase the flow 15 to 20 times per year to handle runoff from the ten to twelve major storms which occur in a year. Flow conditions, of course, were established in anticipation of some rains which did not occur and for some rains which produced less volume of runoff than was expected. With the present average flow in the canal of 3,200 cu ft per sec (1,700 cu ft per sec sewage or domestic pumpage and 1,500 cu ft per sec diversion from Lake Michigan watershed) it would be necessary to increase the flow sixty to ninety times per year if it were not for the Chicago River Control Works. The number of rains in a year in the Chicago area average about 55 at an individual rain-gaging station and about 90 over a composite of all the observation stations in the district.

As previously stated, of the present total average flow of 3,200 cu ft per sec, about 500 cu ft per sec, annual average, is used for flood runoff from the Chicago and Calumet River watersheds, making the average low water flow that will prevail two thirds of the time approximately 2,700 cu ft per sec. This will require a drop of only 6 in. between Lake Michigan and Lockport which will produce a very slight slope indeed (almost flat) and a maximum velocity of 0.7 ft per sec, with less than 0.2 ft per sec in the main Chicago River. This 2,700 cu ft per sec will be composed of 1,700 cu ft per sec of domestic pumpage (sewage or effluent from sewage treatment plants), and of 1,000 cu ft per sec of fresh water diverted from Lake Michigan. This water from Lake Michigan is that deemed "necessary for the purpose of maintaining navigation in the Chicago River, as a part of the Port of Chicago." Of course, a bad sanitary condition, approaching a nuisance, will be produced in the upper reaches of the Lakes-to-Gulf Waterway, with a sluggish current, and with only 1,000 cu ft per sec of fresh water available to dilute the effluent from sewage treatment plants through which have passed the sewage from an equivalent population of 6,500,000. Although this sewage may be treated to an extent comparable to 85% purification, the residue will be practically equal in effect to the raw sewage from 975,000 people, or 15% of 6,500,000.

With the operation of the Chicago River Control Works in conjunction with the manipulation of the Lockport Controlling Works it will be possible to produce a change in flow conditions in the canal and river system in much less time (perhaps two thirds of the time) than that required with control only at the downstream end of the system. The effect of fluctuations of Lake Michigan water levels on the flow in the Chicago River will be eliminated. The water surface in the Chicago River can be held at any desirable predetermined level and definite vertical underclearance of bridges across the river maintained, except during the infrequent times of flood inflow to the river. If the river level is held low to maintain bridge underclearance, in the interest of navigation, there will be additional volume in the river channel to receive runoff from rains, and this may be sufficient to absorb the additional flow produced by many minor rains without any modification of flow conditions. The inflow from heavy storms will necessitate increasing the flow throughout the entire channel. The water surface slope for this condition can be produced more quickly in the future than in the past by lowering the water surface at Lockport while the river level at Chicago is rising from flood inflow. The vertical clearance under bridges across the Chicago River would be lessened temporarily but even this condition can be corrected in a comparatively short time.

The most important advantage from the Chicago River Control Works, and that which may be termed the primary purpose of such works, is the protection that will be afforded against reversal of the Chicago River into Lake Michigan under any condition. Such reversal will be prevented by the automatic closing of the flood gates in case the river level rises higher than that of the lake. Thus the protection of the purity of the water supply of the Chicago region from contamination by sewage is assured, as far as is possible, along the entire shore line of the State of Illinois; and this has been the primary function of The Sanitary District of Chicago for the past half century.

ACKNOWLEDGMENTS

The Chicago River Control Works were constructed by The Sanitary District of Chicago as a project of the Public Works Administration (PWA) and as a part of the sewage treatment construction program made necessary by the decree of the U. S. Supreme Court on April 21, 1930. Acknowledgment is made to Ross A. Woodhull, President of The Sanitary District and William H. Trinkaus, M. Am. Soc. C. E., chief engineer. The project was designed under the direction of the writer by Ralph R. Leffler, engineer of structural design, and constructed under direction of L. B. Barker, M. Am. Soc. C. E., engineer of construction, and Harry J. Bartz, assistant civil engineer. Joshua D'Esposito, M. Am. Soc. C. E., was resident projects engineer, PWA, and U. F. Turpin, M. Am. Soc. C. E., assistant resident projects engineer, PWA. Major Karrick was the U. S. District Engineer at Chicago. The lock and guide walls were constructed by the Frazier-Davis Construction Company, and G. L. Tarleton, Contractor, Inc.; the basin walls and control gates by the Ketler-Elliott Company; and the electrical work was done by the A. S. Schulman Electric Company.

DISCUSSION

A. E. Niederhoff,[3] **Assoc. M. Am. Soc. C. E.** (by letter).—Several phases of the Chicago River Control Works not previously well understood are explained in this paper. The author has given an excellent and reasonably complete account of the history, design, construction cost, and operation of what has been termed "the lowest lock lift in the world."

On December 26, 1939, the writer made it a point to visit the lock at the mouth of the Chicago River to satisfy a growing curiosity as to the construction and framing of the sector gates and the arrangement of operating machinery. Upon arrival, the lock tender volunteered the information that the head on the lock from the lake side was 0.3 ft. However, upon opening the sector gates no water could be observed flowing from the lake into the lock chamber. If there was a head of 0.3 ft at that time, it completely escaped the attention of the writer.

The structures shown in Fig. 1 deserve some explanation as to how they perform the task of separating, "definitely and effectively, the Chicago River from Lake Michigan." The Government breakwater on the east side of the basin and the North Pier on the north side of the basin are not, and were never intended to be, impervious, watertight structures. Fig. 15 shows in cross section the type of breakwater used. The original construction occurred in 1870–1876 and consisted of dumped rock surmounted by a rock-filled timber crib. The top of the breakwater is a more carefully built structure with a concrete cap that, at last reports, was in good condition. In general, the percentage of voids in the rock mass and the permeability of the structure are great. Obviously, as these two sides of the basin now exist, they cannot possibly hold back a head of water for any sustained period. The question is raised as to how these pervious structures are to be converted into a watertight barrier. Certainly the project cannot be considered complete until some further remedial work has been done and the total cost increased accordingly.

The author mentions that several suggestions for protecting the lock walls from impact and rubbing by boats were ruled out because of the fear of small boats tilting when they became caught under horizontal wales or because the usable lock width would be decreased by the width of the protecting element. These arguments are difficult to comprehend because the same problem of protecting lock walls has occurred repeatedly in the 20 years 1920–1940 and has been successfully solved in the structures on the Ohio, Mississippi, Allegheny, Monongahela, and other rivers. The U. S. Army Engineers have found it good practice and a necessary part of the cost of lock construction to armor all corners, ladder recesses, and even longitudinal rubbing strips on the guide and lock walls. Armor has been incorporated in lock walls of modern structures built in the past two decades. Rolled steel sections embedded in concrete are used, and they present a hard, durable surface against which boats can rub without ruining the concrete. The nominal lock-chamber width is frequently increased 1 in. between neat concrete lines to compensate for the fractional part of an inch taken up by the armor.

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[3] Civ. Engr., U. S. Army Engrs., U. S. Engr. Office, Portland, Ore.

The spalled concrete that has been witnessed on the Chicago River lock, although not of structural importance but certainly unsightly, may be caused primarily by impact from boats. However, the appearance of the broken walls is such that there is doubt in the writer's mind that boat impact alone was responsible. Something more elementary during the construction phase may have been a contributing factor. Poor materials and workmanship coupled with lax supervision during construction have resulted in other concrete structures deteriorating within a year in much the same manner that the lock walls have been broken up. The writer is well aware of the practice of gaging the strength and soundness of concrete by crushing "inspector-made" cylinders of representative concrete batches. This is an applicable criterion of partial suitability only when the same care that the inspector uses is duplicated in placing concrete in the forms. Even this guide is not an adequate substitute for good workmanship under strict supervision to obtain concrete capable of withstanding the ravages of temperature range and exposure encountered in Chicago Harbor. Alternate freezing and thawing, ice conditions, and the critical inspection of an interested public at the exposed location of the lock would seem to make it highly desirable to place in the structure the most durable concrete that engineers and technicians can obtain.

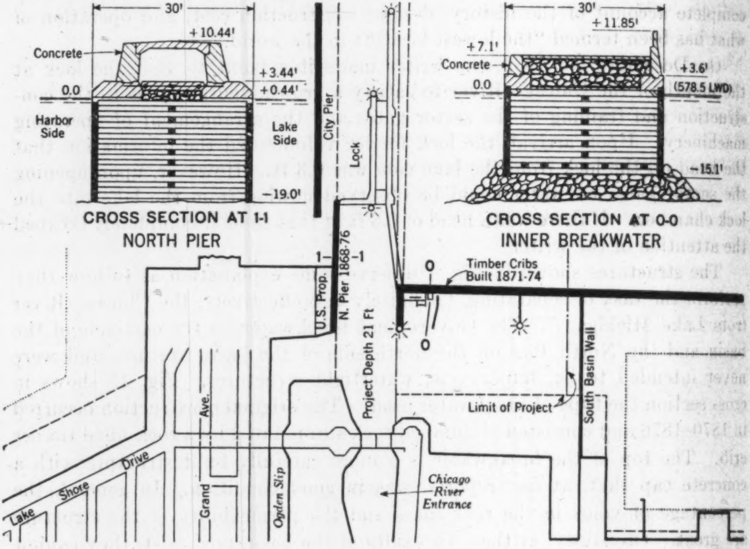
30'
+10.44'
Concrete
0.0
+3.44'
+0.44'
Harbor Side
Lake Side
-19.0'
CROSS SECTION AT 1-1
NORTH PIER
City Pier
Lock
30'
+11.85'
+7.1'
Concrete
0.0
+3.6'
(578.5 LWD)
-15.1'
CROSS SECTION AT 0-0
INNER BREAKWATER
1—1
U.S. Prop.
N. Pier 1868-76
O
Timber Cribs Built 1871-74
O
Project Depth 21 Ft
South Basin Wall
Limit of Project
Ave.
Grand
Ogden Slip
Lake
Shore
Drive
Chicago River Entrance

FIG. 15

The description of the lock gates has cleared up the obvious question of how debris was kept from piling up on the roller track at the bottom of the gate. It is inferred that the combination of wire brooms and raised curbing in which the track is embedded has worked satisfactorily. The operating machinery with its vertical shaft motors and novel method of speed reduction should be described in greater detail. Perhaps an assembly view of the motors and machinery would indicate the desirable compactness of the unit.

Derivation of time required for filling, and of the rate of inflow, for various heads on the lock has been handled particularly well. The reasoning is substantiated by actual tests, and this in itself makes the paper valuable to those engaged in the design of navigation locks. Engineering literature is complete with theoretical analyses for filling various types of conventional locks, and the hydraulic laboratories have reams and reams of data on novel lock-filling and lock-emptying devices. When it comes to actual tests on the prototype, the investigator is able to find but few references. Even these references are almost entirely confined to tests on locks using the conventional through-wall culvert and wall port systems. These tests on filling a lock through sector gates are the first of their kind known to the writer, and the author's diligence and industry in reporting them are to be commended.

R. P. V. MARQUARDSEN,[4] Esq. (by letter).—Mr. Ramey is to be commended for making available to the engineering profession the information contained in this paper. The description is general in character, but the author has managed to go into the various phases of the project in considerable detail, although perhaps not always to the extent that the specialist might desire in connection with the particular line in which he is interested.

Lock Walls and Guide Walls.—Little can be added to the description of the lock walls and the guide walls. The 1-in. mastic expansion joints provided in the reinforced concrete cap of both the lock walls and the guide walls were placed directly above an interlock between two adjacent sheet piles in the face arcs of the cells. To each of these sheet piles was welded, for reinforcement, a 10-in., 45-lb, wide-flange beam, 58 ft in length.

The main features of interest in these walls are:

(1) The arrangement of the diaphragms of the cells, which, as stated by the author, enabled the contractor to start construction at several points. This was a very desirable feature, inasmuch as the existing wave action made it unwise to drive many cells before commencing the work of filling them.

(2) The simple method of construction—the walls were constructed without cofferdams and without employing divers, the type of wall permitting all of the work to be done from above the water level.

(3) The exposed part of the steel sheet piles is permanently below water level, the top being encased in concrete down to low-water level.

(4) The lock walls are so designed that the entire lock chamber may be pumped dry should occasion require.

North Basin Wall.—The cells of the north basin wall were formed by steel sheet piles. All of the piles are 58 ft long and were driven to a bottom elevation of − 55.0, except those for the two end cells and for the diaphragms, which are 48 ft long and were driven to a bottom elevation of − 45.0 Chicago City Datum (C.C.D.). The average penetration of these piles is about 30 ft and 25 ft, respectively. The radius of the arcs of the piles forming the two faces of a cell in the part of the basin wall located north of the basin-wall gate block, or control-gate section, is 34 ft, except for the end cells, where a radius of 18 ft 3¾ in. was used. Two of the diaphragms are straight and two are curved, the radius of the curved diaphragms being 60 ft. The radius of the face arcs of the cell between the control-gate section and the west gate-foundation structure is 45 ft. At the intersection of each diaphragm with the face arcs, special fabricated **Y**-piles were used.

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The north basin wall was designed as a gravity dam in a manner similar to the lock walls, the assumed conditions being as follows: (1) Base of wall, at El. − 26.0 C.C.D.; (2) width of wall at base, 34 ft; (3) water pressure to El. + 6.0 on one side and to El. − 2.0 on the other side; (4) no wave action; (5) weight of rock fill at 90 lb per cu ft in the dry and 65 lb per cu ft submerged; (6) angle of repose of rock fill, 45° (1 to 1 slope), corresponding to a ratio of horizontal to vertical intensity of pressure of 0.172; and (7) no surcharge.

South Basin Wall.—As stated by the author, the **Z**-piles used in the south basin wall were designed as vertical simple beams with a cantilever at the top to carry the horizontal load caused by the submerged rock fill and a head of 7.83 ft of water. The points of support were assumed at El. − 0.50 (the elevation of the horizontal wales and the tie rods) and at El. − 30.11 (the calculated first point of contraflexure below the lake bottom).

The wales were designed as a simple beam to carry the horizontal reaction of the **Z**-piles at El. − 0.50, the points of supports being the tie rods.

The tie rods were designed to take the reaction of the wales on the low-water side of the basin wall, the support of the tie rods being the wales bolted to the **Z**-piles on the high-water side. The **Z**-piles on the high-water side in turn bear against the rock fill of the wall, thus completing the combining of functions of the several parts of the basin wall that enables the wall to act as a gravity dam.

The south basin wall, whose north face is a continuation of the north face of the U. S. Naval Reserve, has greatly increased the docking facilities at that point, the wall forming the south limit of a sheltered basin.

The wall, in conjunction with the United States breakwater, also serves as a vehicular and pedestrian traffic connection link from the U. S. Naval Reserve to the U. S. Coast Guard station and to the south half of the lock.

Lock Gates.—The author mentions briefly the rubber sealing strips provided at the bottom and the two vertical sides of each gate-leaf face. These rubber strips serve to seal the gates when the lock is closed. The vertical edge of each gate-leaf face at the intersection with the recess side closes against a vertical structural-steel support embedded in the gate block. The other vertical edge (at the intersection of the gate face with the lock side) of a leaf closes against the corresponding edge of the companion leaf of the same gate. The bottom edge closes against the cutoff curb on the floor slab of the gate-foundation structure.

The rubber sealing strips at the bottom edge and at the recess-side vertical edge are of the so-called lamb-chop design, being shaped somewhat like a lamb chop with a ⅝-in. hole in the enlarged portion. The over-all width is 4⅛ in.; the general thickness, ¾ in.; and the diameter of the enlarged portion, 1¾ in. The section is reinforced with 4-ply, 18-oz ducking, which envelops an interior body of soft, pliable rubber, the outer rubber encasing the duck reinforcement being tire-tread stock.

The rubber sealing strips at the vertical edge closing against the corresponding edge of the companion leaf of the same gate are of a special design, the general shape being that of an arc having a radius of 1 ft 6½ in. The over-all width of the strips is 15 in. The thickness at the center (for a width of 9 in.) is 2 in., and at the two sides, 1 in. The section is composed of two parts cemented together to form a single unit. The part placed against the structural steel is made of 15-in. belt backing reinforced with 4-ply, 28-oz ducking. The other part is reinforced with 4-ply, 18-oz ducking and covers a 1-in. layer of sponge rubber in the center width, the outer rubber encasing the duck reinforcement being tire-tread stock.

The rubber sealing strips are fastened to the gate leaves by means of ⅝-in. bolts, spaced about 6 in. on centers, and 2-in. by ⅜-in. continuous bars, placed on the outer face of the sealing strips. The rubber sealing strips of the special design are fastened with two rows of bolts and two continuous bars, one at each side.

Concrete Gate Blocks.—Each gate-foundation structure supporting the two leaves of a lock gate, as the author states, is of rigid reinforced concrete construction and consists of two gate blocks (one for each of the two gate leaves) rigidly connected to, and resting on, a reinforced concrete floor slab, the distance between the inner faces of the gate blocks being 80 ft, the clear width of the lock chamber. The floor slab, including its untreated foundation piles and surrounding steel sheet piles, is described quite fully by the author.

Each gate block consists of a certain number of reinforced concrete walls and slabs, so arranged as to provide the required features. The general dimensions and elevations are given by the author. Some of the features common to each gate block are: (1) A gate-leaf recess; (2) an operating-strut chamber; (3) an operator's house; (4) electrical manholes; (5) two tile gages for indicating water levels; (6) connections for two temporary curved cofferdams of single rows of steel sheet piles, extending from gate block to gate block; and (7) two connections for a temporary straight cofferdam enclosing the gate-leaf recess.

The gate-leaf recess, as stated by the author, is shaped like a 60° sector with a radius of 52 ft 6 in. This radius is for the wall of the recess, the radius for a cantilevered curb at the top of the gate block being 50 ft 9 in. The recess is used for housing the gate leaf when the lock is open. The details of the gates are such that the face of the timber wales on the lock side of the leaf is 8 in. behind the face of the gate block when the leaf is housed.

The top of the floor of the operating-strut chamber slopes toward the gate-leaf recess, the elevation varying from + 2.0 C.C.D. at the entrance to + 2.75 C.C.D. at the rear. The top of the roof is at El. + 7.0, except over the operating pinion and the structural-steel carriage where the elevation is + 10.0. The floor is provided with a horizontal concrete runway in which the track plates for the operating-strut casters are embedded at El. + 3.0. The operators' houses are described quite fully by the author.

Two tile gages for indicating water levels are set in the lock-side faces of each gate block, one on each side of the gate-leaf recess. The top of the top marking of the tile gages is at El. + 6.0 and the bottom of the bottom marking is at El. − 8.0, thus allowing for a variation in level reading of 14 ft. Each foot is divided into tenths, the numbers and the graduations being navy blue and the field white. The gages are made of standard bright-glazed ceramic tiles.

The connections for the cofferdams consist of steel piles cast in the gate blocks. They are located in small recesses in the gate-block face and extend vertically from the top of the gate block to the top of the floor slab, one interlock protruding beyond the concrete surface of the recess, for connection with the end sheet piles of the cofferdams.

The two temporary curved cofferdams consisting of single rows of steel sheet piles and extending from gate block to gate block are for use when it becomes necessary to make repairs to both leaves of a gate. One cofferdam is located on each side of the gate and thus they provide, in conjunction with the gate blocks, a complete gate enclosure from which the water may be removed and in which the gate may be repaired in the dry. Each row of steel sheet piles forming a cofferdam is assembled to a radius of 48 ft 1¾ in.

The temporary straight cofferdam enclosing the gate leaf is for use when it becomes necessary to make repairs to a single gate leaf. The cofferdam consists of a single row of Z sheet piles supported at the top against a double structural-steel girder and at the bottom against the concrete curb on top of the floor slab, already mentioned by the author. The cofferdam, when in place, covers the entire lock-side opening of the gate-leaf recess and thus provides, in conjunction with the walls of the gate-leaf recess, a complete enclosure for one gate leaf, from which the water may be removed and in which the leaf may be repaired in the dry.

A special feature for the northeast gate block is a boat stall, 6 ft wide by 10 ft high by 29 ft 3 in. long, the bottom of the stall being at El. − 4.0. The boat entrance of the stall is provided with mooring bars, and with ladder rungs for ingress and egress of the boat operator through a manhole in the floor of the northeast operator's house.

The principal features of the gate-foundation structures, worthy of mention, are:

(1) Each gate-foundation structure, being designed as a rigid structure, insures the uninterrupted functioning of the gate leaves, even if the structure, through settlement at one end or side, should tip slightly as a whole toward that end or side;

(2) The sheet-pile connections in the gate blocks connect the temporary steel sheet-pile cofferdams and provide enclosures for each individual gate leaf or for each gate. The proper combination of these cofferdams can also be made to form a single enclosure for both gates and the entire lock chamber;

(3) The details of each gate leaf recess are such that the gate leaf cannot overrun while closing, the vertical structural-steel support embedded in the concrete work for closing contact with the rubber sealing strips at that edge of the gate face being adjusted to act as an automatic stop when the gate leaf is in a fully closed position; and

(4) The provision in the northeast gate block for a boat stall in which is kept a boat for use in cases of emergency, the boat stall being within the lock chamber when the lock is closed.

Operating Machinery.—The machinery for operating the lock gates is of a very ingenious design and, in the writer's opinion, deserves a more detailed description. The operating machinery for each gate leaf consists essentially of a power unit and an operating unit. The assembly of the operating machinery is shown in Figs. 16 and 17.

The power unit (see Fig. 16) has two motors: A large motor for high-speed operation, which is a 25-hp, 1,160 rpm motor; and a small motor for low-speed operation, which is a 1½-hp, 1,145 rpm motor.

A 350 to 1 reduction (for high-speed operation) between the output shaft of the large motor and the output shaft of the power unit is obtained in two steps as follows: First reduction (5.846 to 1) through a set of planetary helical gears meshing with a pinion on the output shaft of the large motor; and second reduction (60 to 1) through the internal mechanism of a large heliocentric reducer whose input shaft is connected to the output shaft of the planetary helical gears. The output shaft of the heliocentric reducer is also the output shaft of the power unit.

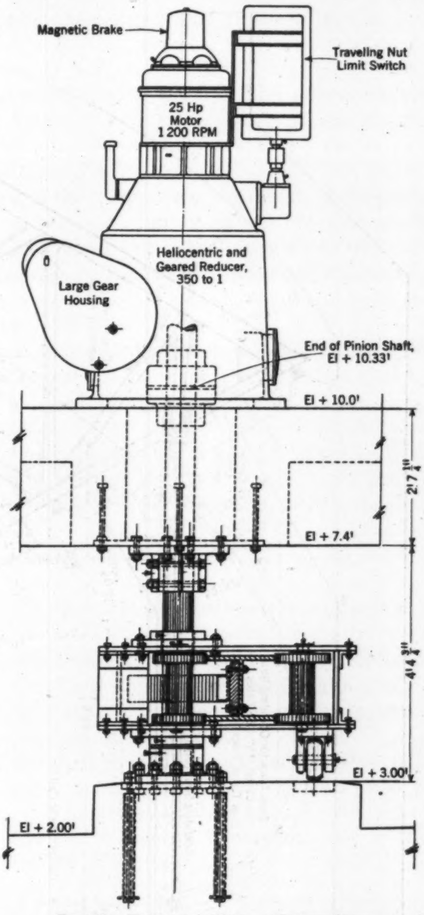

FIG. 16.—ELEVATION VIEW OF POWER UNIT

A 30,000 to 1 reduction (for low-speed operation) between the output shaft of the small motor and the output shaft of the power unit is obtained in four

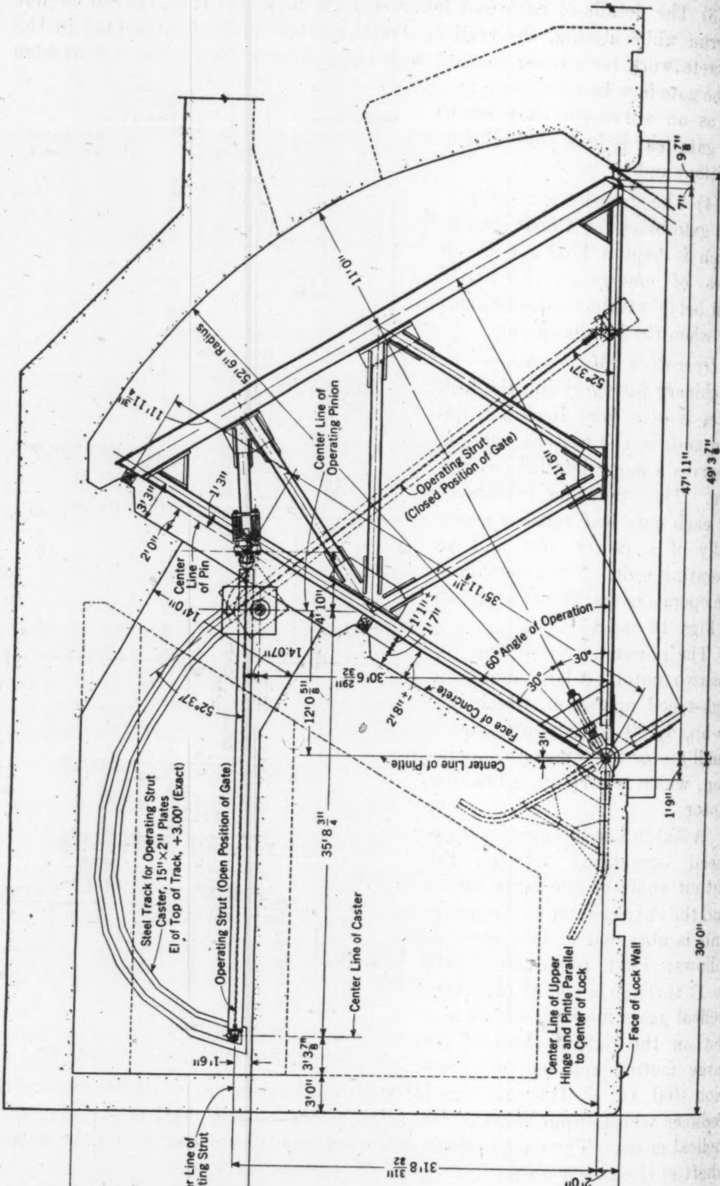
52'6" Radius
11'0"
11'11¾"
3'3"
1'3"
2'0"
Center Line of Pin
Center Line of Operating Pinion
Operating Strut (Closed Position of Gate)
52°37'
41'6"
35'11½"
14'0"
14.07"
4'10"
1'1"±
1'7"
60° Angle of Operation
30°
30°
30'6 29/32"
2'8"±
Face of Concrete
12'0 5/8"
3"
Center Line of Pintle
52°37'
Steel Track for Operating Strut Caster, 15"×2" Plates El of Top of Track, +3.00' (Exact)
Operating Strut (Open Position of Gate)
35'8¾"
Center Line of Caster
3'3 7/8"
1'6"
3'0"
Center Line of Operating Strut
Center Line of Upper Hinge and Pintle Parallel to Center of Lock
Face of Lock Wall
31'8 3/32"
2'0"
15'6"
32'3"
9 7/8"
7"
47'11"
49'3 7/8"
1'9"
30'0"

FIG. 17.—LOCATION OF OPERATING STRUT

steps as follows: First reduction (4.529 to 1) through a small helical gear meshing with a pinion on the output shaft of the small motor; second reduction (40 to 1) through the mechanism of a small heliocentric reducer whose input shaft is connected to the output shaft of the small helical gear; third reduction (4.03 to 1) through a large helical gear meshing with a pinion on the output shaft of the small heliocentric reducer; and fourth reduction (41 to 1) through a worm wheel, which is bolted on the external face of the internal rack of the large heliocentric reducer, meshing with a self-locking worm on the output shaft of the large helical gear. The worm wheel, when in operation, connects with the output shaft of the large heliocentric reducer, which (as stated previously) is also the output shaft of the power unit.

Because of the self-locking effect of the worm drive, the design of the power unit is such that both the large and the small motors may be operated simultaneously. This feature makes it possible to cut in the large motor while the small motor is still operating, which means that the strut load to be picked up by the large motor is smaller than it would be if the small motor had to be cut out before the large motor could be cut in.

If only the small motor is to be operated, it becomes necessary to make the internal mechanism of the large heliocentric reducer inactive. This is done by applying a magnetic brake mounted on top of, and connected to, the large-motor shaft.

The output shaft of the large heliocentric reducer (and of the power unit), to which the vertical shaft of the operating unit is attached by means of a flexible coupling, thus revolves at a higher or lower speed depending on whether the large or the small motor is being operated, the higher speed of the shaft (and of the operating pinion) being 3.32 rpm and the lower speed 0.0382 rpm, for the aforementioned reductions.

A limit switch of the traveling-nut type, providing six sets of contacts and mounted in a cast-iron frame with a gasketed cover, is bolted to the frame of the power unit. Its mechanism is connected to the output shaft of the large heliocentric reducer through a set of 5.5 to 1 helical gears, resulting in a minimum amount of backlash. The vertical shaft of the operating unit is supported in a vertical bearing, bolted to the concrete gate block.

The operating pinion keyed to the shaft is of forged steel and meshes with a cast-steel rack bolted to a horizontal structural-steel operating strut, which is provided with a caster at one end and is hinged to a spring buffer connected to the gate leaf at the other end. The caster runs on track plates set in the concrete gate block, at El. + 3.0 (see Fig. 17).

The spring buffer consists of a cast-steel frame attached to the gate leaf, four steel springs (18½ in. free length) pre-stressed to remain inactive under a tensile or compressive stress of as much as 30,000 lb in the operating strut and held in place by four shafts (passing through the springs), and a two-way hinge casting for connection with the operating strut. The buffer is provided with a limit switch with spring return to trip out the starting contactor of the operating motors in case of excessive stresses in the machinery. The switch, which is mounted in a watertight cast-iron case, is operated by means of a lever moved by the change in position of the spring buffer. One unit of the switch opens when the total compressive stress in the operating strut reaches 45,000 lb and closes when the stress is reduced to normal. The other unit of the switch opens when the total tensile stress in the operating strut reaches 45,000 lb and closes when the stress is reduced to normal.

A structural-steel carriage, which is provided with a caster roller running on a segmental plate set in the concrete gate block at El. + 3.0, turns on the vertical pinion shaft and holds the rack in mesh with the pinion, the rack being kept in proper position by four vertical guide rollers forming part of the carriage. The rollers, two on each side of the operating strut, turn against the top and bottom flanges of the strut to which the rack is bolted.

The sequence of operation of the operating machinery in closing and opening the gates is described quite fully by the author and needs no further elaboration. Attention may be called to the compactness of the power unit, which occupies approximately an area of 4 ft by 6 ft and stands only about 8 ft above El. + 10.0, and to the fact that the operating strut and rack and the operating pinion and structural-steel carriage are all above El. + 4.0, the normal high-water level of Lake Michigan.

Miscellaneous Structures.—Among the many miscellaneous structures and structural devices provided in connection with control works and not referred to by the author the following may be mentioned: (1) Reinforced concrete stairs in connection with the crest walls, (2) ladders, (3) mooring hooks, (4) mooring rings, (5) check posts, (6) capstans, (7) pipe handrailing, (8) manholes and handholes, and (9) life preservers.

Reinforced concrete stairs with pipe handrailing, for getting from one side of the crest walls to the other, are provided at several points, there being two such stairs on the southwest guide wall, three on the south lock wall, and three on the southeast guide wall.

Ladders for descending from, or ascending to, the top of the lock are provided at several points, there being three on the lock-side face of each of the lock walls, five on the southwest guide wall, six on the southeast guide wall, two on the northeast guide wall, and two on each of the four gate blocks. The ladders consist of 1½-in. galvanized-steel ladder rungs set in 1-ft 6-in. by 1-ft 6-in. vertical grooves in the face of the concrete work, from the top of the lock to the bottom of the concrete work, which, in the case of the gate blocks, means to the top of the floor slabs. Ladder rungs of appropriate design are also set in the face of the concrete walls of the gate blocks, at various points.

Mooring hooks for the temporary tying-up of motor boats and other smaller craft while in the lock are provided at several points, there being eight in the lock-side face of the lock walls and two in each of the gate blocks. The mooring hooks are located at El. + 2.0 in an 8-in. deep recess, 1 ft 5 in. wide by 2 ft 3 in. high. The mooring hooks are galvanized and shaped approximately like an S. They are forged from steel rods 2½ in. in diameter.

One mooring ring is anchored at the east end of each of the two east guide walls and one at the west end of the southwest guide wall. The rings have an inside diameter of 6 in. and the eyehook anchor is 2 ft long, both parts being galvanized and forged from 2-in. steel rods.

Check posts for mooring large vessels while in the lock, or while waiting to pass through the lock, are cast in and anchored to the concrete work at El. + 7.0, at about 65-ft intervals on each side of the lock and guide walls (except on the south side of the south lock wall, where there is only one check post at each end) and at 86-ft intervals on the lock side of the gate blocks, there being two check posts per gate block. The check posts are of cast steel and are 1 ft high above the top of the base, which is 1 ft 3 in. square and 3 in. thick. The top of the base is set flush with the top of the concrete, each check post being anchored by means of four 1½-in. bolts, 1 ft 9 in. long. The body of the check posts is 10 in. in diameter at the top and 5½ in. at the bottom.

A hand-power capstan for use in opening or closing the gate leaves manually in cases of emergency is provided at the top of each gate block. Each capstan is anchored securely to the gate block and is provided with four wooden capstan bars for turning.

Pipe handrailing is provided at critical points over the entire lock: (1) On the lake side of the northeast and northwest gate blocks and on the west side of the northwest gate block, (2) around the gate-leaf recess of each gate block, (3) along the north side of the north lock wall and of the northeast guide wall, and (4) around the inside of the walks on each gate leaf. The handrailing consists of two 1½-in. pipe rails and 2-in. pipe posts, connected together and in place with malleable fittings and flanges, the flanges being cinch-bolted to the top of the concrete work. The handrailing is 3 ft 6 in. high, the distance between the pipe rails being 1 ft 6 in. and the pipe posts being spaced about 5 ft on centers. All of the handrailing parts are galvanized.

Electrical manholes and handholes are constructed in the gate-foundation structures and in the reinforced concrete frame of the north lock wall and northeast guide wall, at required points. The manholes and handholes are provided with frames and covers, and two pulling irons are installed in each manhole.

Life preservers in steel cases with glass fronts are provided at 12 points in the lock: Two on each side of the south guide wall, two on each of the lock walls, and one on each of the four gate blocks.

Construction Features.—As stated by the author, no particular difficulties were encountered in the construction of the lock. The steel sheet piles for the several cells of the lock and guide walls were driven in about 24 ft of water into the lake bottom, which is soft gray clay for the top 5 or 8 ft and hard gray clay of increasing stiffness below the soft clay.

Double-acting hammers, on floating rigs, were used for driving. The cells were driven to heavy timber templates. At starting points, the templates were supported, just above water level, by temporary framework consisting of cross-braced wood piles driven into the lake bottom. For succeeding cells, the templates floated on the water level and were held in place by being lashed to the cell previously driven. All of the steel sheet piles of a cell were assembled around the template before driving commenced, and they were all driven completely before work was started on a succeeding cell.

Although the construction of the walls was performed in water partly sheltered from storms, it was found inadvisable to drive too many cells before filling them with stone or gravel. Consequently, the work of driving the steel sheet piles for the cells and the delivery of the stone and gravel for filling the cells had to be coordinated in such a manner as to have a sufficient number of cells ready for filling when the stone or gravel arrived and yet not have them remain unfilled for too long a period.

The stone was delivered by large self-unloader boats, in lots ranging from 2,850 to 7,700 tons. The gravel was delivered in similar but smaller boats, in lots ranging from 1,900 to 4,100 tons. The stone and gravel were unloaded from the boats directly into the cells without rehandling and without any labor being performed in the cells.

Both steel and wood forms were used in the construction of the reinforced concrete frames encasing the top of the steel sheet piles of the lock and guide walls. The outside bottom and side forms were suspended from overhanging structural-steel frames resting on jacks placed on top of steel columns supported in the stone or gravel fill of the cells. Four steel columns were used in each cell, one in each corner. They were left in place and encased in the concrete frame. No bottom forms were used for the concrete work inside the cells. The side forms rested on the stone or gravel fill.

All concrete for the lock and guide walls and for the two gate-foundation structures was mixed at, and distributed from, a "pump-crete" plant located on top of the cofferdam for the west gate-foundation structure, at the southwest corner. The cement, sand, and gravel were delivered on, and used from, barges tied to pile clumps alongside the mixing plant, there being no material storage on the cofferdam except very small quantities for use in cases of emergency.

The upper hinge of each gate leaf was set and concreted in its gate block before the lower hinge was placed. The position of the pintle of the lower hinge was established by plumbing from the upper hinge. The lower hinges were set and concreted in recesses left in the floor slabs of the gate-foundation structures.

Two divers were employed in placing the pre-cast reinforced concrete floor slabs for the lock chamber. To minimize the required slab movement at the lock-chamber bottom, the location of the slab joints were marked on the walls (above the water level), and the boom of the crane was set to correspond before the slabs were lowered into the water. After the crane had lowered the slabs to almost the lock-chamber bottom, the divers had little more to do than to square up the slabs just as they were being released.

ISAAC DEYOUNG,[5] M. AM. SOC. C. E. (by letter).—Works of the magnitude and importance of the Chicago River Control Works should be so built that there will be no likelihood that operations will be interrupted. It is unfortunate that ample funds were not available to permit the construction of the Chicago River Control Works in accordance with designs ordinarily used for such structures. There is no inexpensive method of constructing works of this nature and magnitude for permanence.

The established plane of reference of Lake Michigan is 578.5 ft above mean tide at New York City. The elevation of Chicago City Datum (C.C.D.) is 579.94 ft above sea level. The floor of the lock is at El. 24.94 C.C.D. and is therefore 23.5 ft below the plane of reference of Lake Michigan. The sheet piles of the lock wall cells were driven to − 55 C.C.D., or about 30 ft below the floor of the lock. The lock walls were designed to sustain a head of 29 ft of water. Although the lock will be rarely unwatered, should this be done there would be, theoretically, an uplift pressure of about 1 ton per sq ft under the floor of the lock. It is not believed that the unbalance of pressure will cause any concern, as the piling has been driven to the stiff clay stratum. However, since the material is plastic, it is believed that it would have been better if the floor of the lock had been made of reinforced concrete with a transverse sectional form of an inverted arch and anchored to the clay bottom by timber piling. It is certain that, in time, a considerable volume of debris will be carried into the lock chamber by the action of vessels' propellers. If the floor of the lock were made in the form of an inverted arch, the collection of the debris would be largely confined to the low parts of the floor and could be removed more easily by a derrick boat with clamshell bucket. Deep recesses near the gates are of a great advantage for the collection and confinement of debris.

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It is an established fact that much material and debris are moved by the action of the propellers of vessels. Shoals are formed to a considerable extent at the approaches, as well as in the lock, and will require removal to maintain project depth.

It is noted that in less than one season of use there has been considerable spalling of the concrete faces of the lock. Since a concrete top is attached to the steel piling of the cellular walls of the lock, it is believed that the use of pressure concrete, shot by a nozzle, will give a mixture of much greater strength than form-placed concrete. The compressive strength of such a mixture is from 6,000 to 10,000 lb per sq in. and is double that of ordinary form-placed concrete; less mass will be needed and no forms will be required.

To protect the present lock-wall faces from abrasion and further spalling, oak fenders may be attached to them, a three-strake oak fender being bolted to two ship channels fastened to the masonry with fox bolts. The oak fenders are placed in the troughs of the channels and the center oak strake is placed between the top and bottom channels. The maneuvering of vessels cannot be controlled to any extent.

Broken rock was preferred for the filling of the cells because of greater shear resistance. Gravel filling would be more compact and free from voids. It is believed that if timber piles were driven within the cells in conjunction with the gravel filling it would provide the desired shear. The added cost of the piling would, in part, be offset by the lower unit cost of gravel. Settlement of the rock filling will continue for a long time.

An excellent pavement may be made on the top and back of the piers and lock walls by using a mixture of tar and gravel, topped with road asphalt and covered by a thin layer of pea-sized gravel, the pavement being built about 3 in. deep. The tar and gravel are mixed in a small concrete mixer, using a mixture of 3 gal of tar and 7 cu ft of gravel. This mixture is spread over a leveled surface and compacted by tamping or by a hand roller. After rolling, the surface is covered with heated road asphalt, over which a thin layer of pea-sized gravel is spread.

In the matter of underwater lubrication of bearings, where low temperatures exist, the selection of a proper lubricant is important. It is found that a grease with a lead soap base has given excellent service. Chemically prepared graphite greases or oils have not given satisfactory service; the liquid vehicle is lost, leaving the graphite to solidify in the grooved bearings, thus preventing the lubricant from functioning. Because of the low temperatures, it requires considerable pressure to force the lubricant to the bearings.

JOHN W. WOERMANN,[6] M. AM. SOC. C. E. (by letter).—An excellent and detailed account of the design and construction of a new type of navigation lock has been presented by Mr. Ramey. It is particularly adapted to construction in deep water where a head is likely to develop at either end of the lock. This paper is particularly valuable on account of the widespread interest in this unusual lock. Letters of inquiry concerning various features of the lock have been received by the U. S. Engineer Office at Chicago from many other U. S. Engineer Offices in the United States.

Credit for suggesting the use of steel cells filled with rock for lock walls and guide walls, thus eliminating the necessity for an expensive cofferdam in 25 ft of water, is due Brig.-Gen. Max C. Tyler, M. Am. Soc. C. E., then assistant to the chief of engineers, U. S. Army. In connection with the design of this lock, General Tyler also suggested a hydraulic study, which was made under the direction of the writer, to consider the economics and feasibility of maintaining an artificial elevation of water surface in the Chicago River and connecting waterways of − 2.0, − 4.0, and − 6.0, Chicago City Datum (C.C.D.). This study showed that the − 2.0 project was the cheapest to construct and maintain. It was also the safest, considering the old wooden pile foundations along the river, because the water in the Chicago River had been at this elevation repeatedly, due to natural causes, for as long as six months at one time, without damaging the buildings that rested upon these wooden piles. The maintenance of an artificial water surface at − 2.0 C.C.D. would furnish additional bridge clearances for the nagivation of river barges and towboats which will be particularly valuable if a system of fixed bridges is adopted at some time in the future. The maintenance of the water surface at − 2.0 also furnishes very valuable potential storage capacity for the temporary storage of heavy runoff which is likely to pile up in the Chicago River several times a year while the hydraulic gradient is being increased between Chicago and Lockport, where the storm water would discharge into the Des Plaines River.

Referring to this artificial water surface of − 2.0, the author (see heading "General Conditions") states that "the differential in water levels may average 2 ft," which is true. However, at times of high lake level, this difference may amount to 5.0 ft, and on rare occasions to 6.0 or 7.0 ft. The author's following statement, "At present this differential is about 6 in., or less," means that at the time the paper was written the difference in level at the lock was 6 in., or less. Usually it is much less, because no effort will be made to establish a water surface of − 2.0 below the lock until the old Government breakwater and the U. S. North Pier are made watertight.

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[6] Cons. Engr., Chicago, Ill.

Stone was used in the lower part of the lock wall cells to reduce the internal pressure in the cells. This is important if it becomes necessary to pump out the lock. The filling consists of rock up to elevation − 2.0 C.C.D., as stated by the author (see heading "Walls: Lock Walls"). The gravel fill extends from − 2.0 to the top of the cell, which is at elevation + 7.0, a total distance of 9.0 ft.

The failure of one of the steel cells in the cofferdam during construction is ascribed, by the author, to the longer radius used in the arcs of the cofferdam cells and the omission of weep holes in the piling. These were no doubt contributory causes, but the character of the filling was undoubtedly a third cause. This consisted of clean, smooth, globular pebbles, about the size of marbles, which had an exceedingly flat angle of repose.

The author states (heading "Walls: Lock Walls"): "The advantage of the open concrete grill on top of the rock-filled cells, acting as a horizontal truss, over the same grill stiffened by a concrete slab is not apparent to many structural engineers. The difference in action, if any, should be very slight." The principal advantage, as seen by the writer, is the saving of concrete. The areas left open during concrete construction and subsequently filled with gravel and topped with limestone screenings constitute a very considerable portion of the total horizontal surface, as will be realized from an inspection of the plan view in Fig. 3. Furthermore, if there should be any settling of the rock or gravel filling, additional material can be added through these openings.

Lock Gates.—With the adoption of the steel cell design for the lock walls, search was started for a satisfactory solution for the lock gates. As stated by the author, "The requirement that these gates be capable of withstanding a head of water in either direction and also remain closed against surges from the lake eliminated ordinary miter gates from consideration." It was necessary also that the lock be filled and emptied from the ends of the lock instead of through culverts in the lock walls in the usual way. Consideration was given to the use of two pairs of mitering gates at each end of the lock, one pair pointing toward the lake and one pair pointing toward the land, but this was considered too complicated and expensive. Consideration was given also to the use of a single reversible pair at each end of the lock (without a miter sill) so that the gates could be pointed in either direction, but this was considered too uncertain and dangerous. Other forms were given consideration, including sector gates of the type used in the Blue Island lock on the Calumet-Sag Channel. This type admits water only between the gates, along the center line of the lock, and creates considerable disturbance to a tow in the lock.

A more desirable type of sector gate was found in the new lock at Södertälje on the Södertälje Canal, about 42 km (26 miles) southwest of Stockholm, Sweden. This lock is described in detail in a report prepared by Sal. Vinberg and Lars Lawski, Swedish Government engineers, for the Thirteenth International Congress of Navigation held at London in 1923.[7] In describing the lock and some of the conditions that had to be met, Messrs. Vinberg and Lawski state that, although the lock gates are required to resist only a small head, this head is exerted sometimes from one side and sometimes from the reverse side. Another condition to be provided for was the escape through the lock of a large quantity of water during the spring floods. The locks accommodate ships drawing from 21 to 23 ft of water, carrying 6,000 to 8,000 tons when fully loaded. The water depth on the sills is 24 ft 7 in., the height of the gate 32 ft 1.73 in., the width of opening 65.62 ft, and the clear length of the lock, 442.91 ft.

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[7] "Dispositions or arrangements to be adopted for locks, inclined planes, and other means of overcoming differences of level with a view to facilitating the operations," by Sal. Vinberg and Lars Lawski, *Report No. 13* relating to Inland Navigation, XIIIth International Cong. of Navigation, London, 1923.

Consideration was given to the use of roller gates, double mitering gates, reversible mitering gates, locked mitering gates, bascule gates, and sector gates. Referring to the sector gate, Messrs. Vinberg and Lawski say that it is a requirement for the best and most continuous filling of the lock that the greater part of the inflowing water should pass around the gates. At Södertälje two thirds of the water is made to pass through the gate recesses. The water that flows through a gate recess is forced against the wall of the recess and runs along it as if it were flowing through a side channel. The three streams coalesce close below the gates, and thus both their kinetic energy and their velocity are eliminated, and the water flows smoothly and with but small velocity into the lock chamber.

After comparing the advantages and disadvantages of each of the six types of lock gates enumerated above, the authors conclude that, taking into account the lack of security in operation of locked gates and reversible mitering gates, and the high cost of construction and operation of double mitering gates, it is obvious that the final decision will be in favor of either sector gates or roller gates. Of these two, the sector gates are the cheaper; the operating procedure is simpler, as there are no sluice gates to operate; operating the gates is less complicated; the time required for filling and emptying the lock is shorter, giving increased capacity to the canal; the security in working is greater than with the other types of gates, as sector gates can be operated in running water; and, on account of the strong and stable attachment to the masonry, ice and wind will have less effect upon them. For these reasons, it was decided to have sector gates for the lock at Södertälje. A plan sketch of the gates is shown in Fig. 18.

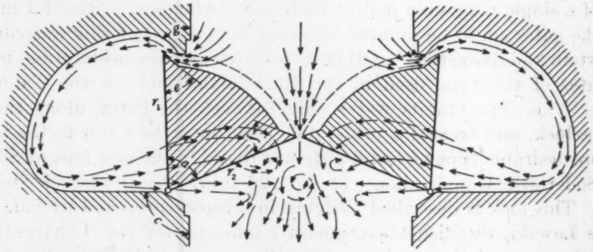

FIG. 18

As this type of gate was new, models were made to determine the best value for the center angle at the hinge of the gate (angle *a*, Fig. 18); the gate opening speed; the time required to take a boat through the lock with various lifts; the distribution of the inflowing water in regard to force and direction;

stresses in the cables with which the boats were moored (parallel to the longitudinal direction of the lock); and other features. It was found: That, taking into consideration the cost, the center angle (a) should be 70°; that the downstream wall of the gate recesses should be at right angles to the face of the lock wall (angle c, Fig. 18); that the angles e, f, and g, Fig. 18, should be alike and equal to 20°; that the gate radii, r_1 and r_2, should be approximately equal; and that about one third of the inflowing water should pass between the gates and about two thirds through the gate recesses. Other characteristics were determined which are not of general application.

It was originally intended to place a driving gear on the wall, which would mesh into a rack along the outside plate of the gate, but on account of the possibility of unequal settlement in the masonry, and of vibrations in the gate, wire rope transmission was substituted. This construction also decreases the strain on the machinery on account of the elasticity of the wires and the spring connections between the ends of the wire rope and the gate. The springs become tense when the gate comes into its final position, and in this way help to overcome the inertia in opening the gate. There are two different motors working on equalizing gears, thus allowing the drum to be driven at different speeds.

The results obtained in these model studies of the sector gates at the Södertälje lock have been summarized briefly because they constitute the basis of the design adopted for the sector gates of the Chicago River lock.

The author's summary of the important features of the Chicago River lock gates; his description of the concrete gate blocks, operating machinery, electrical features, and sluiceways; his cost data, hydraulics of filling and emptying the lock; and his statement of benefits from the control works—are all extremely complete, interesting, and valuable.

EDWARD SOUCEK,[8] JUN. AM. SOC. C. E. (by letter).—This discussion is confined to the hydraulic calculations in that part of the paper headed "Operation of Lock." The writer has recently had to study a similar problem and was particularly interested in the formulas and coefficients used in the paper.

The author's formula for discharge is a form of the "Weisbach" equation without velocity adjustments. This equation has been used for calculating the obstructive effect of bridge piers but is now commonly considered to be theoretically incorrect for that purpose.[9, 10, 11, 12] It has been shown,[13] however, that if no drop-down occurs upstream from, or within, the contraction, it is proper to use the Weisbach equation. Accordingly, the author's choice is believed to be correct.

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[8] Associate Hydr. Engr., The Panama Canal, Balboa Heights, Canal Zone.

[9] "Obstruction of Bridge Piers to the Flow of Water," by Floyd A. Nagler, *Transactions*, Am. Soc. C. E., Vol. LXXXII, December, 1918, p. 334.

[10] "Calculation of Flow in Open Channels," by Ivan E. Houk, M. Am. Soc. C. E., *Technical Reports*, Part IV, Miami Conservancy District (1918), p. 262.

[11] "Pile Trestles as Channel Obstructions," by D. L. Yarnell, *Technical Bulletin No. 429*, U. S. Dept. of Agriculture (1934).

[12] "Bridge Piers as Channel Obstructions," by D. L. Yarnell, *Technical Bulletin No. 442*, U. S. Dept. of Agriculture (1934).

[13] "Experiments on the Flow of Water through Contractions in an Open Channel," by E. W. Lane, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919-1920), p. 1149.

By adopting the Francis formula for the "weir" term rather than the customary $\frac{2\,b}{3}\sqrt{2\,g}\,h^{1.5}$, the author has in effect applied the coefficient 0.62 to the weir term as usually written. When he applies an over-all coefficient of 0.95, the final formula is equivalent to the Weisbach equation with a coefficient of 0.95 for the orifice term and a coefficient of 0.59 for the weir term. It would seem preferable to examine the formula in this manner because the element of crest contraction—probably the essence of the value 3.33 in the Francis formula—is entirely lacking in the present case.

There is some evidence that a lower coefficient should be used for the weir term than for the orifice term in the Weisbach equation. An extensive series of experiments to determine the discharge between Chanoine wickets[14] has been performed by the U. S. Engineer Department in the University of Iowa Hydraulics Laboratory. It is stated, in the reference cited,[14] that some confirmation of these tests by field measurements has been obtained. Coefficients in the Weisbach formula were found to vary with the degree of rounding of contraction edges, the width of the openings, and the ratio of depths upstream and downstream from the contraction. The coefficients increased as the depths became more nearly equal. This variation supports the author's choice of a lower coefficient for the weir term.

One objection to the use of the Weisbach equation (or any other containing more than one term) is the complexity of the resulting equations for filling and emptying time. For the range of heads covered by Fig. 13—up to 6 ft—the weir term, although by no means negligible, is not large in comparison with the orifice term. Exponential approximations to the discharge equations may be used to simplify the integration and expedite calculation. Restricting the argument to heads not exceeding 6.0 ft and values of d between 18 ft and 30 ft, Eqs. 3 are replaced, respectively, by the expressions

$$Q = 7.5\,b\,d\,h^{0.44} \qquad \text{(12a)}$$

for filling, and

$$Q = 7.75\,b\,d\,h^{0.53} \qquad \text{(12b)}$$

for emptying, the lock.

The differential equation $Q\,dt = -A\,dh$ may be integrated between the limits h_1 and h by inspection, yielding, as a result, expressions which replace the logarithmic Eq. 4a and the trigonometric Eq. 4b—namely,

$$T = \frac{1{,}450}{\sqrt{d}}\sqrt{h_1^{0.56} - h^{0.56}} \qquad \text{(13a)}$$

for filling, and

$$T = \frac{1{,}560}{\sqrt{d}}\sqrt{h_1^{0.47} - h^{0.47}} \qquad \text{(13b)}$$

for emptying, the lock. In the case which has been worked out by the author, Eqs. 13 reduce to forms which replace Eqs. 8 and 11, respectively,

$$t = 277\sqrt{h_1^{0.56} - h^{0.56}} \qquad \text{(14)}$$

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[14] "Discharges through and over Chanoine Wicket Dams," *The Experiment Station Bulletin* (Hydraulics), U. S. Waterways Experiment Station, Vol. I, No. 3 (1938), p. 4 (condensed from a report of the U. S. Engineer Sub-Office (unpublished), Hydraulics Laboratory, Iowa City, Iowa, 1938.

for filling and

$$t = 337 \sqrt{h_1^{0.47} - h^{0.47}} \ldots\ldots\ldots\ldots\ldots (15)$$

for emptying. These expressions can be solved very rapidly with a slide rule and give results for all purposes equivalent to the transcendental forms used by the author. To demonstrate this, Table 2 has been prepared from Eqs. 14 and 15 for comparison with Fig. 13.

TABLE 2.—FILLING AND EMPTYING TIMES, IN MINUTES

Head, in feet	Initial Head, in Feet; Filling						Initial Head, in Feet; Emptying					
	6.00	5.00	4.00	3.00	2.00	1.00	6.00	5.00	4.00	3.00	2.00	1.00
6.0	0.00						0.00					
5.0	2.40	0.00					2.45	0.00				
4.0	3.50	2.53	0.00				3.55	2.58	0.00			
3.0	4.33	3.60	2.57	0.00			4.50	3.76	2.76	0.00		
2.0	5.17	4.60	3.86	2.85	0.00		5.45	4.87	4.13	3.08	0.00	
1.0	6.10	5.58	5.00	4.27	3.17	0.00	6.47	5.95	5.40	4.63	3.46	0.00
0.5	6.60	6.20	5.60	5.00	4.10	2.56	7.14	6.70	6.13	5.50	4.57	2.98
0.0	7.62	7.25	6.80	6.28	5.58	4.61	8.53	8.20	7.80	7.30	6.60	5.62

In comparing Table 2 with Fig. 13, it must be kept in mind that a 9-in. total width of openings represents 1 min of time. Calculations for the table were made with a slide rule and are not necessarily correct in the third significant figure.

Aside from obvious advantages in calculation, particularly for preliminary designs, it is believed that inspection of Eqs. 14 and 15 gives a better idea of the filling and emptying operations than does examination of the transcendental forms, despite the empirical basis of the former. It should be emphasized, however, that the use of these exponential approximations is merely an analytical trick, possessing no significance whatever beyond the study of the case at hand.

In conclusion, a word of commendation for the good comparison between the author's theory and the test operations is in order. The agreement is indeed excellent. It appears that no allowance was made for flow under the gates when they were not in contact with the sill against which they sealed. In some cases, particularly with an elevated sill, this flow might be quite considerable. It would be interesting to know if this flow was negligible or if the coefficients in the discharge equations were chosen a little high to allow for it.

W. C. WEEKS,[15] M. AM. SOC. C. E. (by letter).—The description and illustration of the design and construction of the new navigation lock in the Chicago River present a masterpiece of lucid engineering presentation. Building a durable and serviceable structure of this magnitude within a reasonable time, and, more particularly, for materially less than the public funds appropriated therefor, is not common in recent years. The accomplishment reflects unusual and well-deserved credit upon the engineers of The Sanitary District of Chicago and their assistants.

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[15] Col., Corps of Engrs., U. S. Army (*Retired*); Dist. Engr., Jacksonville, Fla.

The new lock is intended primarily as a safeguard to the water supply of the City of Chicago. For this purpose it appears to be admirably adapted. Incidentally, its construction, and that of essential appurtenant structures, provides for positive control of the diversion from Lake Michigan, maintenance of the level of the Chicago River below that of the lake, and desirable anchorage areas. With respect to navigation, however, the lock appears to be an impediment, inasmuch as every craft entering or leaving the river must use the lock. This involves a delay of at least 15 min for each lockage.

Mr. Ramey's paper doubtless has been studied in connection with the one by Major Karrick,[2] who recounts, in comprehensive manner, the difficulty of the city in maintaining the purity of its water supply ever since it had one, and that of The Sanitary District to the same end. With respect to the prevention of lake pollution, mention is made of suggested methods other than a navigation lock. None of these, however, were acceptable to all concerned, but the objections were not specifically set forth.

One of the suggested methods which seems to merit special mention contemplated closure of the river during intervals of flow reversal by means of a pontoon gate hinged at one abutment. This plan, if adopted, apparently would have solved the problem with respect to lake pollution. It would have afforded a navigable pass 220 ft wide instead of an 80-ft lock chamber. Probably its interference with navigation in general between lake and river ultimately would be much less than that due to lockage. Finally, its cost would be much less than that of a lock.

From the record, with a diversion of 5,000 cu ft per sec, the pontoon would be closed from 10 to 12 times per year. Such closures would prevail over maximum periods of 9 hr; the average probably would not exceed 7 hr. Chicago city bridges are not operated for navigation during rush traffic hours. Closed bridges are an absolute bar twice daily to such craft as cannot pass beneath them. Relatively few lake vessels use the river, and the bulk of navigation consists of tugs and barges. For this purpose, the pontoon gate appears to offer material advantages. Interchange of lake-borne and river-borne cargoes over the North Pier, if developed for such purpose, will reduce the number of lockages. However, civic attitude toward commercialization of the lake front probably will minimize, and may entirely prevent, such development.

Among the advantages claimed for the lock is the ability to lower the level of the Chicago River. This provides greater clearances under the bridges, a reservoir for moderate runoffs, and facilitates its prompt discharge. Lowering the river level is not necessarily limited to 2 ft below lake level as now contemplated. Demands for additional clearances may arise. If granted, the stability of many structures adjacent to the river would be endangered seriously. Installation of a pontoon gate would have forestalled such demands.

Traffic on the upper reaches of the Lakes to the Gulf Waterway is borne on the raw sewage of a city of a million people. The volume of this sewage two thirds of the time is 2,700 cu ft per sec. The maximum current is about 0.67 ft per sec; that in the Chicago River is almost negligible; and in the pools above the locks it is just about nil. From a sanitary standpoint, these are not navigable waters.

Obviously, the less the diversion from Lake Michigan, the more often the pontoon gate would be closed and the greater its hindrance to navigation. At hearings before a special master, the late General Edgar Jadwin, M. Am. Soc. C. E., then chief of engineers, testified in effect that a diversion of 5,000 cu ft per sec from Lake Michigan would be necessary to safeguard navigation on the Chicago River. It is believed that the soundness of this testimony will be verified during the observation period now presumably in effect. If so, a pontoon gate with respect to Chicago River navigation appears advantageous.

However, the new lock is a reality, apparently satisfactory to all concerned, and for this reason should be accepted as the best solution for the problems involved. With respect to cost, it is an unusual accomplishment. Ingenious modification and adaptation of standard types of construction effected material savings with no sacrifice of permanence, stability, or usefulness. Particularly noteworthy is the design of the lock gates for the special conditions to be met. Taken in its entirety, it is an example of genuine engineering. The exercise of careful thought and planning is evident throughout. To effect so material a saving in the expenditure of public funds nowadays is conspicuous for its rarity.

It is believed that the engineering profession, particularly with respect to the younger members engaged on "projects" financed by public money, has suffered serious deterioration in recent years. Many of these projects have been politically conceived, located, and constructed. Economic justification in very many cases has not been established. It also is believed that the technical press has not fulfilled its mission adequately with respect to publicly financed engineering undertakings. A fearless and forthright presentation of such undertakings from an economic standpoint would have been of great educational value. Such presentations rarely have been made. The general public should not have to depend for its enlightenment upon nontechnical writers in current magazines as to how, where, and why its money is spent.

Mr. Ramey's contribution to the profession will be sincerely—even gratefully—appreciated by all engineers who believe in the homely old adage that "An engineer is a man who can do with one dollar what any fool can do with two." To such, it is as an oasis in a desert.

HENRY R. KING,[16] ESQ. (by letter).—An unusually novel feature of the Chicago River lock is the triangular-shaped gate leaves, providing three passages for the admission of the water when the gates are in a partly opened position, in place of one central passage if true sector gates had been used. Separation of the inflow into three streams tends to dissipate the kinetic energy of the incoming water with reduced turbulence within the lock. This is particularly true if, as in the case of the Chicago lock, the two streams entering the gate recesses are deflected back toward each other impinging on opposite sides of the stream flowing through the central passage. The suitability of the gates for low head locks was convincingly demonstrated in the test of June 25, 1938, when the lock was filled from an initial head differential of 8 ft. Very little turbulence occurred in the lock during the filling operation.

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[16] Senior Civ. Engr., The Sanitary Dist. of Chicago, Chicago, Ill.

The effect of various factors on the time required for operation of the lock may be more apparent if Eqs. 4a and 4b are changed to the form for filling:

$$T = \frac{5{,}480}{C^{0.5}\,R^{0.5}\,d^{0.25}}\left[\log\frac{1.31\,d^{0.5}+h_1^{0.5}}{1.31\,d^{0.5}-h_1^{0.5}} - \log\frac{1.31\,d^{0.5}+h^{0.5}}{1.31\,d^{0.5}-h^{0.5}}\right]^{0.5} \quad ..(16a)$$

and for emptying:

$$T = \frac{5{,}550}{C^{0.5}\,R^{0.5}\,d^{0.25}}\left[\tan^{-1}\left(\frac{h_1}{2.40\,d}\right)^{0.5} - \tan^{-1}\left(\frac{h}{2.40\,d}\right)^{0.5}\right]^{0.5} \quad ..(16b)$$

in which R = the rate of increase in the combined width of the three openings, in inches per minute; C = the coefficient of contraction; and the other symbols are as indicated by the author.

During a filling or an emptying operation the discharge at any particular head is proportional to the total width of the three openings, which varies as $R\,T$. The time, however, varies inversely as $R^{0.5}$, and therefore the discharge must be proportional to $R^{0.5}$. For example, in Fig. 13 the maximum rate of discharge into the lock when filling from El. − 3.0 to + 3.0 is 1,120 cu ft per sec. If the opening rate at low speed were increased four times, the maximum rate of discharge into the lock would be $4^{0.5} \times 1{,}120 = 2{,}240$ cu ft per sec, and the time required for equalization of levels would be $\frac{1}{4^{0.5}} = 0.5$ths as long.

The test of June 25, 1938, was a good check of the accuracy of the formula for computing the filling time. Since the time varies inversely as the square root of the contraction coefficient a reasonable error in its selection would only slightly alter the results.

The author's formula for discharge into the lock may be derived in another way if it is assumed that a horizontal submerged plate at the elevation of the water surface in the lock extends from the openings out into the lake. Then the water entering from above this plate would flow through the openings at critical depth and that from below would be the true orifice discharge, and the formula would be

$$Q = C\left[\frac{2}{3}\,b\,h\left(2\,g\,\frac{h}{3}\right)^{0.5} + b(d-h)(2\,g\,h)^{0.5}\right] \ldots\ldots\ldots(17)$$

Letting $C = 0.95$, this reduces to

$$Q = 7.62\,d\,b\,h^{0.5} - 4.69\,b\,d^{1.5} \ldots\ldots\ldots\ldots\ldots(18)$$

which corresponds to Eq. 3a; and for normal operating conditions the results will not differ from the author's value by more than 0.5%.

From the standpoint of turbulence in the lock the kinetic energy of the incoming water is of more importance than the rate of inflow. This is expressed as $\frac{W\,v^2}{2\,g}$ in which W = weight in pounds. The kinetic energy of the discharge of Eq. 17 is as follows:

$$K = C\left[\frac{2}{3}\,b\,h\left(2\,g\,\frac{h}{3}\right)^{0.5} \times 62.5\,\frac{h}{3} + b(d-h)(2\,g\,h)^{0.5} \times 62.5\,h\right] \quad ..(19)$$

in which K = kinetic energy of the inflow per second, in foot-pounds. Simplified, the formula becomes

$$K = C\,[501\,d\,b\,h^{1.5} - 437\,b\,h^{2.5}] \quad \ldots\ldots\ldots\ldots (20)$$

For the 6-ft fill from El. − 3.0 to + 3.0 (see Fig. 13) the maximum kinetic energy of the inflow occurs at a discharge rate of about 960 cu ft per sec and is 239,000 ft-lb per sec. The kinetic energy at the maximum rate of inflow of 1,120 cu ft per sec is only about 73% as great, or 173,000 ft-lb per sec.

Triangular-shaped gates have certain advantages, particularly in entering the water into the lock in three streams which result in more effective dissipation of its kinetic energy. However, the flow of water into the gate recesses carries with it debris and, in cold weather, ice, which may lodge there and interfere with gate operation. Inasmuch as these were problems to be worked out from operating experience it will be of interest if the author discusses just what modifications, if any, are now contemplated in the gates and just what difficulties have been encountered in this respect.

H. P. Ramey,[17] **M. Am. Soc. C. E.** (by letter).—The writer wishes to express his appreciation to those who have discussed this paper and have thereby called attention to certain items which were perhaps inadequately described.

Mr. Niederhoff has called attention to the fact that the Government breakwater and North Pier, forming enclosure walls of the Chicago River Control Works project, were not impervious, watertight structures. The rock-filled timber cribs forming these structures offered so little resistance to the flow of water from the lake to the river that it was impossible, except under conditions of heavy flow, to maintain water levels in the river appreciably lower than in the lake. During the summer of 1939 a careful survey was made of the Government breakwater and North Pier to determine the extent of work necessary to render these structures watertight, and on August 15, 1940, a contract was awarded for such work. The contract provides for driving a row of steel sheet piling 10 ft from, and parallel to, the landward face of the cribwork; tying the sheet piling to the concrete cap over the cribs, with steel tie rods at proper intervals; filling the 10-ft space between cribs and sheet piling with clay (from Chicago subway excavation); and protecting the row of steel sheet piling with stone fill (on the landward side) to the top of the piling. This work was completed in November, 1940, and added $127,000 to the $2,704,000 cost mentioned in the paper.

Messrs. Niederhoff and DeYoung have both mentioned the spalling of concrete, from impact by boats, and both have made excellent suggestions. Armor, wales, and fenders were considered, but all were ruled out at the time because of cost. It is the conviction of the writer that armor and either wales or fenders will be installed ultimately. The plans were reviewed by a PWA Review Board, interested in keeping costs within an allocated sum. The concrete is of good quality. In many places where concrete has spalled, the particles of coarse aggregate (granite gravel) have split as readily as the remainder of the concrete. There has been no crumbling. The placing of this concrete was inspected by the Sanitary District, the PWA, and representatives of the Army engineers.

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[17] Asst. Chf. Engr., San. Dist. of Chicago, Chicago, Ill.

Mr. Marquardsen, who was one of the principal structural designers on this project, has contributed a valuable discussion of many of the details of the work. His discussion of the operating machinery has probably answered a question suggested by Mr. Niederhoff.

Mr. DeYoung has suggested that a monolithic concrete floor would have been better than the loose slab floor, particularly if the lock should be unwatered. The writer agrees with this view. Mr. Leffler, who designed the lock, favored such a concrete floor, placed "in the dry," to assure accurate construction and to prove the stability of the lock walls.

Mr. DeYoung prefers gravel fill for the cells because of its compactness and quick settlement. His suggestion of timber piles within the cells, to provide shear, is very good. Mr. Woermann obviously prefers stone fill, an honest difference of opinion. Mr. Woermann has reviewed some of the considerations given to various types of lock gates, which ultimately led to the adoption of the triangular-shaped sector gates. The distribution of water currents shown in the tests of the model of this Swedish installation checked conclusions reached by the engineers of the Sanitary District.

Mr. Soucek's discussion of the hydraulic formulas is interesting and constructive. His simplified formulas will give results sufficiently accurate for any practical purpose. He points out that the adoption of the Francis formula for the "weir" term may not be correct since the element of crest contraction is entirely lacking. Apparently he is of the opinion that the product of the weir coefficient 0.62 and the over-all coefficient 0.95 conforms closely to that obtained in tests. The coefficient 0.95 as used in the formula, however, was introduced to take care of side contractions. Probably more in line with what Mr. Soucek had in mind is the derivation of the formula as presented in Mr. King's discussion in Eq. 17. Incidentally, Mr. King is entitled to much of the credit for deriving and checking the formulas for discharge to and from the lock. No allowance was made for flow under the gates when they are in a partly opened position. The head differential is so small by the time the gates have opened sufficiently wide to clear the bottom sill substantially that the flow beneath the gate is practically negligible as compared to the flow through the three main orifices.

Mr. King mentioned the accumulation of ice and floating debris carried into the gate recess, between the straight face of the gate leaf and the curved face of the recess. One plan suggested to avoid this difficulty is to attach falsework to the front of the gate leaf in the shape of a circular arc fixed horizontally, with its center at the gate hinge, and extending vertically down to about 4 ft below the water line, or below the line of floating ice. This would obstruct the upper parts of the side orifices and slightly lengthen the time of filling and emptying the lock. No conclusion has been reached on this matter as yet. Ice troubles have been avoided by frequent operation of the gates for the specific purpose of breaking up the ice.

Colonel Weeks, who was United States District Engineer in Chicago during the years when the Chicago River Control Works project was the subject of discussion and reports in the controversy regarding lake levels between the State of Illinois and other Great Lakes states, has contributed a summary of certain events which preceded the adoption of the present plan. His views of the advantages and disadvantages of this structure in Chicago Harbor are sound.

It is extremely gratifying to the writer to have been able to present a paper which has elicited such constructive criticism and so much friendly comment. The labor has not been in vain.

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TRANSACTIONS

Paper No. 2112

MINIATURE SYSTEM OF FIRST-ORDER ALINEMENT AND TRIANGULATION CONTROL

BY FLOYD W. HOUGH,[1] M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. WILLIAM BOWIE, EARL O. HEATON, GEORGE H. DELL, CHARLES B. STANTON, C. L. GARNER, AND FLOYD W. HOUGH.

SYNOPSIS

An unusual type of survey at the Tygart Dam, on the Tygart River, 2 miles upstream from Grafton, W. Va. (Fig. 1), is described in this paper. Completed early in 1938, the structure was built by the Corps of Engineers, U. S. Army (Pittsburgh (Pa.) District) and is the largest concrete gravity dam east of the Mississippi River, containing 1,250,000 cu yd of concrete. The spillway section rises 207 ft above the stream bed and has a maximum capacity

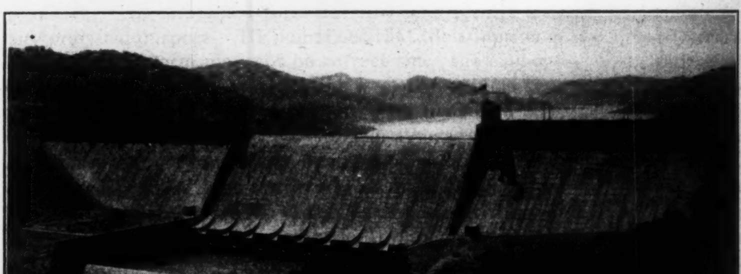

FIG. 1.—TYGART RIVER RESERVOIR DAM AT GRAFTON, W. VA.

NOTE.—Published in December, 1939, *Proceedings*.

[1] Geodetic Engr., U. S. Coast and Geodetic Survey, Washington, D. C.

of 210,000 cu ft per sec. Flanking the spillway section are non-overflow sections 23 ft higher. The total length of the dam at the top is 1,921 ft.

Tygart Dam is the first unit to be completed of the flood-control system for the protection of Pittsburgh and the upper Ohio Valley. It also serves to aid navigation during the dry season on the Monongahela River (to which the Tygart River is tributary) by the release of impounded flood waters.

The method of horizontal alinement control described herein is intended not to replace, but to supplement, triangulation. As the word implies, alinement is pertinent to line rather than area control, the latter being the particular function of triangulation. Within its sphere of adaptation, the alinement method is capable of producing a greater accuracy of relative horizontal position than is the highest grade of triangulation. A survey of this type involves the study and determination of possible small horizontal movements in a fixed engineering structure, such as a large dam, as well as a study of contributing factors in the immediate vicinity. It is felt that, in such a survey, a combination of an accurate alinement survey across, and on an extension of, the structure itself, together with control of the surrounding area by first-order triangulation, furnishes a most appropriate basis for analytical study and for arrival at definite conclusions.

The purpose of the work described in this paper is to develop a suitable means for measuring, from time to time, the direction and amount of any horizontal movements that may occur in the various monoliths of the dam to supplement the data obtained from the strain gages installed in the dam, and to determine ground movements in the surrounding area. Such movements may be caused by deformation of the bed due to reservoir load, by load on the dam, temperature changes, isostatic disturbance in the vicinity of the dam, or by a combination of such forces.

The horizontal control at the Tygart Dam may be divided into two main features—alinement and first-order triangulation.

ALINEMENT

The first-order alinement survey consisted of running straight lines across the dam, at a low stage of the reservoir pool, both longitudinally and transversely. These lines are held by small punch holes in bronze T-bars grouted flush into each monolith. The bars, ½ in. by 3½ in., at the exposed surface, were set with their long dimension at right angles to the alinement line.

The complete alinement plan and the profiles of the longitudinal lines are shown in Fig. 2. Two longitudinal lines, approximately 9 ft apart, were established across the dam, one for the alinement on the wing monoliths and the other for the spillway monoliths. The limiting clearance due to the special design of the pier houses made these two lines necessary. In addition to the alinement bars in the dam, twelve substantial concrete monuments, weighing 800 lb to 1,000 lb each, were established to hold these longitudinal alinement lines on the adjacent ridges. The main line extended to ridges 2.5 miles east

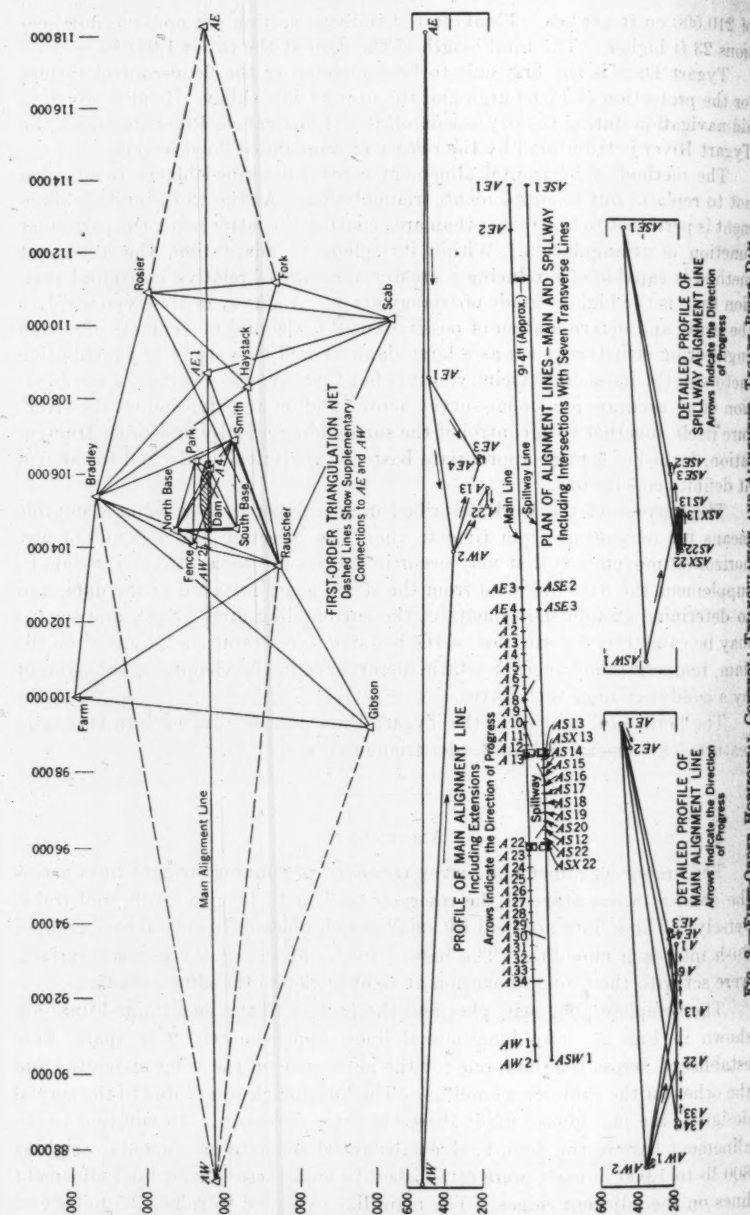
FIRST-ORDER TRIANGULATION NET
Dashed Lines Show Supplementary Connections to AE and AW
Main Alignment Line
Farm
Bradley
Rosier
Fork
Scab
Haystack
Smith
Park
AE1
North Base
South Base
Dam
A4
Fence
AW2
Rauscher
Gibson
AE
AW
100 000
98 000
96 000
94 000
92 000
88 000
90 000
92 000
94 000
96 000
98 000
100 000
102 000
104 000
106 000
108 000
110 000
112 000
114 000
116 000
118 000
PROFILE OF MAIN ALIGNMENT LINE
Including Extensions
Arrows Indicate the Direction of Progress
Elevation, in Feet
1 600
1 400
1 200
AE
AE1
AE3
AE4
A13
A22
AW2
AW
PLAN OF ALIGNMENT LINES—MAIN AND SPILLWAY
Including Intersections With Seven Transverse Lines
Main Line
Spillway Line
9° 4′ (Approx.)
AE1
ASE1
AE2
AE3
AE4
ASE2
ASE3
A2
A3
A4
A5
A6
A7
A8
A9
A10
A11
A12
A13
AS13
ASX13
AS14
AS15
AS16
AS17
AS18
AS19
AS20
AS12
AS22
ASX22
Spillway
A22
A23
A24
A25
A26
A27
A28
A29
A30
A31
A32
A33
A34
AW1
AW2
ASW1
DETAILED PROFILE OF SPILLWAY ALIGNMENT LINE
Arrows Indicate the Direction of Progress
ASE1
ASE2
ASE3
AS13
ASX13
AS22
ASX22
ASW1
DETAILED PROFILE OF MAIN ALIGNMENT LINE
Arrows Indicate the Direction of Progress
AE1
AE2
AE3
AE4
A1
A6
A13
A22
A33
A34
AW1
AW2
Elevation, in Feet
1 600
1 400
1 200

FIG. 2.—FIRST-ORDER HORIZONTAL CONTROL; TRIANGULATION AND ALINEMENT, TYGART RIVER DAM

and 3.5 miles west of the dam. Reference monuments at right angles were set except where the alinement marks were in pairs. Distances to reference marks were measured carefully by means of a light-weight standardized tape. The six transverse alinement bars were fixed in the monoliths where the various triangulation lines crossed the dam.

A 9-in. direction theodolite was used on this work (see Fig. 3). The telescope optics for surveys of this nature should be able to provide a high resolving power which, combined with a high magnifying power of 50, will allow the observer, in pointing, to differentiate between the center of the image and any eccentric fringes caused by atmospheric distortion of the light. The light was centered between two vertical hairs, about 35.″0 apart in the optical center of the telescope. Although a $\frac{1}{8}$-in. slit actually subtends an angle of 2.″0 at a distance of 1,000 ft, the light from this slit on an average night will occupy a width of from 4.″0 to 8.″0 in the telescope hairs, due to diffusion in the 1,000 ft of atmosphere. Experienced observers will agree that a signal can be centered between two such hairs more accurately than a single hair can be pointed upon the signal.

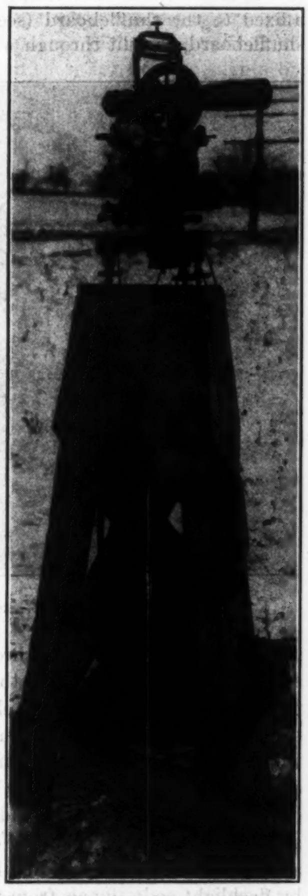

FIG. 3.—NINE-INCH DIRECTION THEODOLITE WITH MOUNTED STRIDE LEVEL

The observations were made entirely at night to minimize adverse atmospheric conditions. Conditions that might cause lateral refraction were avoided. The targets consisted of small electric signal lamps operating on shuffleboards. The lamps and boards were designed in the Surveying and Mapping Section of the U. S. Engineer office, at Pittsburgh, and were machined commercially to detailed specifications. Similar equipment and methods were used successfully in 1932 and 1933 by the geodetic personnel of the Metropolitan Water District of Southern California, in extending the final alinement lines for the 90 miles of tunnels of the Colorado River Aqueduct.[2]

Each signal lamp consisted of an ordinary two-cell focusing flashlight, rigidly set in a small box. The unit cells were removed and the housing rewired to allow the use of 1.5-volt ignition cells with the standard flashlight bulb. The light rays emitted from the lamp were confined to a $\frac{1}{8}$-in. vertical slit in a metal slide over the face of the lamp. At the base of the slit, and accurately centered, was a vertical hairline scratch extending down to the base of the metal frame and riding on the graduations of a section of steel tape affixed to the shuffleboard (see Fig. 4). Since, in moving the light on the shuffleboard, the slit through which all light rays passed traveled horizontally in a vertical plane passing through the alinement station and lying at right angles to the alinement line, the effect of errors of eccentricity in the bulb or the reflector was practically eliminated. Although a narrower slit of light could have been used, it is doubtful if the accuracy of pointing the telescope would have been increased thereby. Each setting of the lamp could be read easily to the nearest 0.001 ft. No tangent screw assembly was included in the design of the board as it would have been impractical in use and would have added nothing to the accuracy of the work. The lamp was set independently at each observation by sliding it on the tracks of the shuffleboard as directed by flashlight code signals from the observer's assistant.

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[2] "Geodetic Engineering Applied to the Colorado River Aqueduct," by Floyd W. Hough, *Military Engineer*, March–April, 1934.

FIG. 4.—SIGNAL LAMP AND SHUFFLEBOARD

Where the alinement bar was less than 1,000 ft from the instrument, a small metal scale, graduated to $\frac{1}{20}$ in., was fixed by adhesive tape directly to the flat surface of the alinement bar. The target now became the point of a sharp needle scriber whose point was moved along the scale. The scale readings were then recorded for each observation, as was done with the steel tape on the shuffleboard.

The total number of observations (varying from 10 to 108) taken on any one alinement bar depended upon the range of readings and upon the distance intervening between the instrument and the bar. The extreme range of readings obtained was closely proportional to the distance from the instrument to the bar and amounted to about 0.01 ft per 300 ft. The two master control points on the main and spillway alinement lines of monolith 13 were observed upon three and two different nights, respectively. Although these bars were 1,500 ft from the instrument, the means of the three night observations on A13 differed by a maximum of only 0.002 ft, and those for the two nights on AS13 by 0.004 ft (see Fig. 2). An abstract of the field work, listing the results of observations on different nights of the two master alinement points, is shown in Table 1.

Given experienced personnel and adequate instrumental equipment, there are many vital details involved in the methods used, the neglect of any one of which may make the difference between first-order results and those of a lower grade of accuracy. Every possible precaution was used at Tygart Dam to eliminate constant and systematic errors and to minimize accidental errors. Among these points were the following:

TABLE 1.—MEAN BOARD READINGS ON DIFFERENT NIGHTS OF THE TWO MASTER ALINEMENT BARS IN MONOLITH 13

Date, 1938; April: (1)	Theodolite at station: (2)	Foresight (3)	Alinement bar* (4)	Number of observations (5)	Mean reading on board, in feet† (6)	Distance alinement bar from theodolite, in feet (7)	Punch hole south of north end, in inches (8)	Punch hole north of south end, in inches (9)	Length of bar, in inches (10)
11	AW2	AE1	A13	28	29.531	1,500	....	....	....
12	AW2	AE1	A13	40	29.529	1,500	....	....	....
13	AW2	AE1	A13	40	29.531	1,500	1.27	2.25	3.52
12	ASW1	ASE1	AS13	30	29.515	1,500	....	....	....
13	ASW1	ASE1	AS13	50	29.511	1,500	1.54	1.95	3.48

* A13 is on main alinement line in monolith 13; and AS13 is on spillway alinement line in monolith 13.
† Range, 3 nights, on A13 = 0.002 ft; and 2 nights, on AS13 = 0.004 ft.

(*a*) Rigid lumber tripod set in ground and loaded with rocks (see Fig. 3).

(*b*) Instrument protected by observing tent.

(*c*) Accurate centering of the instrument within 0.001 ft was accomplished by using new 14-oz plumb-bobs, shielding the bob from air currents, spinning the bob to determine any eccentricity of its point, and having the instrument so oriented that the vertical plane containing the two sections of plumb-line leading from the hook coincided with the vertical plane of the alinement line.

(*d*) The theodolite was adjusted particularly for level, collimation, verticality of the pair of vertical hairs, and parallax.

(*e*) The program was so arranged that alinement points were set by observing on relatively long foresights, obviating the necessity for plunging the telescope between the pointing on the fixed light and the alinement bar to be set. This method tends to diminish any error in alinement as the instrument is moved forward to succeeding stations.

(*f*) Five observations were taken with the telescope in a direct position, followed immediately by five in the reversed position. This program of equal number of direct and reversed positions of the instrument operates to eliminate collimation errors, focusing errors of the draw tube, and errors due to the horizontal and vertical axis not being exactly at right angles to each other. At each observation, the fixed foresight light was purposely thrown out of the hairs by a small movement of the horizontal tangent screw, thus making each observation the result of an independent pointing on the fixed foresight. After setting the movable light between the hairs, the observer, before giving the "O.K." signal, habitually rested his eye by closing it momentarily and then glancing back for a check on the pointing.

(*g*) A sensitive stride level on the horizontal axis was used continuously throughout the observations instead of depending on the relatively sluggish plate level. The error due to the inclination of the vertical axis cannot be eliminated or reduced by the reversal of the instrument. An accurate stride level spanning the full length of the horizontal axis, and properly operated, is the only satisfactory answer. The stride level itself was kept in careful adjustment. Its own remaining error of adjustment was eliminated by lifting it from the horizontal axis and reversing it end for end during reversal of the instrument. The effect of this was to throw the small angle of adjustment error equally on opposite sides of the vertical plane of the alinement line. It should be noted that this took care only of the lack of perfection in the adjustment of the stride level itself. To eliminate the systematic effect of vertical axis inclination and transform it to a small compensative error, the stride level was brought to center by the instrument foot-screws after each reversal and, when necessary, immediately prior to each pointing on the fixed foresight.

(*h*) Personal error in the setting of the target on the scale was made compensative by bringing up the target alternately from the left and right. The observer at the instrument could not see the scale readings, and the person shifting the target was not in a position to see the readings on the scale until after receiving the observer's "O.K." signal.

(*i*) When using the needle scriber for a target, the illumination was alternated from equal angles to right and left of the scriber point to eliminate the error of phase.

(*j*) All readings of the target scales were checked independently by the recorder and entered in ink in the record book. The shuffleboard, which stood about 7 in. above the alinement bar, was weighted down securely. However, to furnish a positive check on the stability of the board, readings of both ends of the bar were taken on the tape both before and immediately after the observations. All computations and data in the record book were checked and initialed by both the target operator and the recorder.

(*k*) After the mean reading was determined, it was plumbed carefully the few inches from the scale on the shuffleboard down to the bronze bar and a small hole punched in the bar. In a case where the small scale was in contact with the surface of the alinement bar, the hole was punched at the mean before removing the scale. The two short distances from the punch hole to either end of the bar and the length of the bar, for a check, were entered and initialed in the record book so that if the punch hole should become worn or even removed entirely, it may be replaced with complete assurance in its original location on the bar.

When using the aforementioned equipment and observing the precautions outlined, the alinement work can be done more accurately than even double first-order triangulation, due to the fact that neither the errors of the graduated circle nor of the micrometers are involved in the alinement observations.

From an analysis of the agreement between observations taken on different nights and the probable errors of the observations involved, it is apparent that a horizontal movement of 0.01 ft on the dam as a whole in respect to the adjacent ridges, or of half that amount of relative movement among the monoliths, can be shown conclusively by the alinement procedure described.

TRIANGULATION

The first-order triangulation scheme consisted of sixteen stations arranged in an overlapping system of figures, concentrated mainly in the vicinity of the dam but extending east and west across valleys to adjoining ridges. The triangulation of the main scheme covered a small area of only 3.3 sq miles, with lines from 440 to 7,800 ft long, the median line being 3,450 ft. The reconnaissance was done with a particular view to proper strength of figure and avoidance of lines subject to lateral refraction.

Stations were marked by bronze plates in large concrete monuments poured in place and set 4 ft into the ground. The excavation was enlarged toward the base. This provided a batter to the monument for resisting frost action along the sides. The stations were referenced by two other concrete monuments.

The angles were observed with the direction theodolite shown in Fig. 3, with sixteen positions of the circle on each of two nights. Any directions differing by more than 4″.0 from the mean for that night were rejected and a new mean taken. Observations were recorded in ink on special mimeographed forms. Computations on these record sheets were checked and initialed carefully.

The small lamps previously described, with ⅛-in. vertical slits, were used as targets. The same care in centering the instrument and lamp was used on the triangulation as in the alinement work. Except for the problem of centering and the necessity for the use of an accurate stride level because of steeper vertical angles, it is scarcely more difficult to obtain good triangle closures on short lines than on long ones. However, considering that an eccentricity of 0.005 ft will cause an error of 1″.0 on a line 1,000 ft long, and proportionally greater or less errors for shorter or longer lines, respectively, it is clear that the problem of centering becomes acute. That this problem may be taken care of satisfactorily is evidenced by the following statistics from the simultaneous least-squares adjustment of the first-order triangulation net at Tygart Dam:

Probable error of an observed direction	±0″.47
Average correction to a direction	0″.38
Average closing error of a triangle	1″.07
Maximum correction to a direction	1″.43
Second largest correction to a direction	1″.03
Maximum closing error of a triangle	2″.77
Mean error of an angle	±0″.74
Average correction to an angle	0″.62
Maximum correction to an angle	2″.41
Number of triangles	31
Number of plus closures	18
Number of minus closures	13
Lengths of line, in feet:	
Maximum	7,800
Minimum	440
Median	3,450

There were several nights without a single rejection. The rules for rejection of observations listed in the manual of first-order triangulation[3] were rigidly followed throughout the field and office work.

The stride level was employed continuously on all observations throughout the triangulation, using the same detailed methods of operation as on the alinement. Before each half position, the stride level was centered by the instrument foot-screws with the telescope pointed approximately in the direction of the light having the greatest vertical angle. An azimuth from Polaris was observed to orient the triangulation.

A base 1,650 ft long was measured with three different 50-m invar tapes, each of which had been certified as to length, weight, and coefficient of expansion by the National Bureau of Standards. The base extended over rough ground, but the grade did not exceed 9% in any tape length. The three measurements of the base with the three different tapes fell within a range of 1.0 mm, which resulted in a computed probable error of 1 : 2,500,000.

After the lists of directions were checked and initialed, the triangulation was adjusted rigidly by the application of the method of least squares. The main scheme adjustment involved thirty-one triangles, thirty condition equations (including twenty-one angle and nine side), and sixty-eight unknowns. The relatively large number of condition equations was due to overlapping figures. The additional observed lines were introduced to strengthen the net by increasing the number of conditions to be satisfied in the adjustment. Stations AE1, AW, and AE (Fig. 2) were fitted to the main scheme by secondary adjustments.

Rectangular coordinates of all triangulation stations were computed on a horizontal plane through the crest of the dam. They were based on an assumed origin and the observed true azimuth at station FARM.

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[3] *Special Publication No. 120*, U. S. Coast and Geodetic Survey.

Further elaboration as to detailed methods of triangulation, base measurement, and computations, is not made in this paper because they conformed to standardized procedure for first-order geodetic surveys already published[4] in various treatises on this subject.

SUMMARY

The use of first-order triangulation to determine small earth movements has been employed extensively on the west coast by the U. S. Coast and Geodetic Survey. More recently the triangulation has been supplemented by alinement lines. These have been extended across known fault lines to measure small longitudinal movements.

The particular method of alinement surveys used on the Tygart Dam and for the tunnels of the Colorado River Aqueduct, as previously mentioned, has a limited field of application; but within that field it represents a most accurate means of laying a straight line on the topographical surface and of determining at a later date any changes from that line.

The "straight line" here obtained is in reality the alinement curve on the spheroid modified slightly by the deflection of the plumb-line, the latter being due to the effect of local topographic masses. This alinement curve lies between the two plane curves connecting the ends of the line and, due to the short distances between instrument stations, very nearly coincides with the geodetic line connecting the end points. In fact, for the short distances involved, all of these curves may be considered as coincident straight lines. Even in large triangulation figures, the observed direction, which is a plane curve, is treated as a geodetic line.

The deflection of the plumb-line, due to the different effects of topographic mass attraction at the various stations along the alinement line, undoubtedly does cause some deviation from the desired line; but even if it were shown that these effects were not entirely negligible from a practical standpoint, they still would not detract from the accurate determination during future surveys of movements at right angles to the alinement line. This is true because the deflection of the vertical, which is the small angle between the normals to the geoid and the spheroid at any point, may be assumed to remain constant, and the original line would be rerun under identical conditions as to topographic mass attraction, except for the changes in the reservoir mass whose effect would be negligible.

The system of overlapping triangles in a net of first-order triangulation (such as that established in the vicinity of the Tygart Dam to detect any small earth movements which might occur in the surrounding ridges) creates numerous conditions which must be fulfilled exactly to secure proper correlation between the ground points. The theory of least squares, by which the most probable values are obtained, lends itself admirably to the solution of these various condition equations.

The high order of accuracy obtained in the horizontal control at the Tygart Dam, together with the stability and permanence of the marked stations, furnishes an ideal condition for economical remeasurements at any future time to determine what small changes, if any, have occurred, both in amount and direction.

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[4] *Special Publications Nos. 28, 120, 138,* U. S. Coast and Geodetic Survey.

Although certain portions of the continent are known to be more stable (geologically speaking) than others, geologists will agree that it cannot be stated with assurance that any part of the earth's crust is entirely free from the possibility of isostatic adjustments at any unforeseen time. With this in view, does it not seem logical that 0.03% of the cost of an engineering structure, such as a large dam, may reasonably be devoted to a first-order control survey which will show conclusively just what movement, if any, is taking place in the surrounding ground as well as on the structure itself?

It is felt that, with the rapidly increasing appreciation for the economic advantages of more accurate surveys based on definite standards rigidly adhered to, such as those adopted by the International Geodetic Association and the U. S. Coast and Geodetic Survey, the time is not far distant when it will become common practice to determine the ground position of important engineering structures by the use of properly established and adjusted first-order control.

ACKNOWLEDGMENTS

W. E. R. Covell, M. Am. Soc. C. E., Lieutenant-Colonel, Corps of Engineers, U. S. Army, is district engineer of the Pittsburgh District. The work described in this paper was done under the supervision of the writer in the Surveying and Mapping Section headed by Payson A. Perrin, Assoc. M. Am. Soc. C. E. The instrumental observations were made by John P. Mapes, junior engineer. The direction theodolite (see Fig. 3) was loaned to the District by the Director of the U. S. Coast and Geodetic Survey.

DISCUSSION

William Bowie,[5] M. Am. Soc. C. E. (by letter).—No doubt methods similar to those used by Mr. Hough in making an alinement survey and a triangulation at Tygart Dam, as a means by which horizontal movements of the land near the dam and distortions of the dam itself can be detected, will be used at others of the many dams that have been erected in recent years. That knowledge of the degree of stability of a dam structure and of the surrounding area is desirable admits of no doubt. The problem confronting engineers has been the devising of a method by which this could be done. Mr. Hough has solved the problem successfully.

The apparatus used by him in the alinement survey is ingenious; of particular interest is the lamp described. Engineers wishing to conduct alinement surveys and triangulation should profit by Mr. Hough's experience and make their observations at night. The officials of the U. S. Coast and Geodetic Survey also learned that observations for triangulation could be made more rapidly, accurately, and economically at night than during daylight.

In order to do good work on such a project as that which Mr. Hough describes, the best of instruments should be used. It is wasteful to use transits and poor direction theodolites since finer instruments are now on the market and can be purchased at reasonable costs. Engineers who are satisfied with only the most up-to-date machines and other equipment seem to be loath to obtain high-grade surveying instruments and retire the old-fashioned ones.

The exceedingly high accuracy secured by Mr. Hough in his triangulation is noteworthy, especially since his lines were short. One familiar with the textbooks on higher surveying of the past generation will recall that primary triangulation (now designated first-order) required triangle sides that were 10 miles or more in length. In fact, triangulation of primary, secondary, and tertiary character referred more to the lengths of lines than to accuracy.

For years the officials of the U. S. Coast and Geodetic Survey have advocated the use of geodetic methods and instruments on even comparatively small survey jobs. Any engineer who really knows a surveyor's transit and the micrometer microscope used in measurements in a physics laboratory can obtain acceptable results, after a few hours' practice, with a good direction theodolite. With correct eyesight and more experience he can obtain excellent results.

Geodetic surveying such as Mr. Hough describes is high-class engineering. What he accomplished is a challenge to other engineers which it is hoped they will accept.

Earl O. Heaton,[6] M. Am. Soc. C. E. (by letter).—It was gratifying to read Mr. Hough's excellent paper and to note at what a small percentage of the cost of the construction project it was completed. In demonstrating how small the cost of an accurate survey really is, this paper should be helpful in persuading other construction executives, in public and private enterprise, to execute surveys of this type.

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[5] Hydrographic and Geodetic Engr., U. S. Coast and Geodetic Survey (*Retired*), Washington, D. C. Died August 28, 1940.

[6] Hydrographic and Geodetic Engr., U. S. Coast and Geodetic Survey, Washington, D. C.

A point which Mr. Hough did not mention, but which seems very important, is the possibility of the future use of this miniature system of control for expansion of the present project, or for other new engineering and construction projects in the immediate vicinity. Mapping and surveying of any nature that might seem desirable can then be started from this existing miniature system.

Instruments and methods for the triangulation described by Mr. Hough are similar to those which were used by the writer on first-order triangulation and traverse control surveys of Rochester, N.Y., and Atlanta, Ga. On these projects it was also found that short lines do not make accurate observations difficult, provided reasonable care is used in centering lights and instruments.

In making the aforementioned surveys, small lamps similar to those described by Mr. Hough were used on traverse lines much less than 1,000 ft long. Although the lamp slit was much wider than ⅛ in., the additional width of opening did not affect the accuracy of the observations adversely. Great care was exercised, however, to eliminate possible eccentricity in the reflector and bulb, and this was accomplished by placing a piece of thin paper or ground glass over the slit.

GEORGE H. DELL,[7] ASSOC. M. AM. SOC. C. E. (by letter).—The application of modern first-order methods to the establishment of precise reference lines for the study of horizontal displacements in large structures is ably described in this paper by Mr. Hough. The process involves two major steps of equal importance—namely, monumentation and observational procedure.

It is evident that the monumentation system of the Tygart Dam alinement has been carefully planned so as to permit the lines to be re-established at any future time even if some of the monuments become disturbed. Each of the two alinement lines extends over a considerable distance and is permanently monumented at six points, the monuments being supplemented by transverse reference monuments. The triangulation system, the lines of which intersect the alinement lines at varying angles, serves to provide a means of determining the directions and magnitudes of probable earth movements at various points. The fact that the dam design was such as to require a separate line for the spillway alinement has apparently resulted in a strengthening of the system. Since the two lines were established independently, the monuments which were installed at a number of opposite points serve not only as transverse reference marks for the detection of future disturbances but provide additional checks on the original alinement observations. This feature of double alinement would appear to be desirable in any case. In this connection, it occurred to the writer that it might have been advantageous to have alinement bars installed on the spillway line opposite those placed on the main alinement. It is probable that such a system would provide additional aids in detecting alinement errors due to possible disturbances to the monuments and in determining differential displacements of points on the monoliths.

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[7] Associate in Civ. Eng., Univ. of Illinois, Urbana, Ill.

The paper treats the details of observational procedure with due consideration to their importance, and it should serve as a valuable guide to engineers who are called upon to execute similar projects.

At the same time, engineers who are faced with the problem of fixing relatively accurate alinements with limited instrumental facilities will find many useful suggestions in the paper, such as having the instrument in good adjustment; shielding the instrument and plumb-bob from the effects of wind; centering over the points with care; leveling the instrument with the aid of the telescope bubble; setting points on line, when possible, by sighting on a distant foresight; using a shuffleboard or a suitable substitute for obtaining the readings; and guarding against phase errors. Points which have to be set by plunging should later be rectified by a series of direct observations. It is also important that the bearings which support the horizontal axis be free from looseness or irregularities due to wear.

The exact sequence that was followed in establishing the points on the Tygart Dam alinement does not seem to be entirely clear; it would be helpful if the author would elaborate on this phase of the problem in his closing discussion.

CHARLES B. STANTON,[8] M. AM. SOC. C. E. (by letter).—In his excellent, although somewhat brief, paper Mr. Hough has presented a simple procedure for the precise determination and recording of the location of structures in reference to the surrounding terrane, and also for the establishment of permanent reference points on the main structure which can be used in the future for determining any movement of the component parts of the structure.

More thought should be given, by those in charge of the construction of projects involving a change in the use of a terrane, to the possibility of the isostatic adjustments incident thereto and to making provisions for the determination of the amount of such changes.

In the paragraphs preceding "Triangulation" the author presents eleven precautions to eliminate constant and systematic errors and to minimize accidental errors. These items contain the key to success for any survey of this type. By carefully outlining the procedure followed in the work, Mr. Hough has brought to the attention of his readers the necessity for maintaining a constant alertness in order to eliminate all systematic errors and to prevent, as nearly as possible, the entrance of accidental errors into the work.

The details of procedure as outlined in paragraphs (*f*) and (*g*) of the "precautions" should be considered with care by all engaged in similar work. These paragraphs outline procedures whereby all errors incident to the instrument construction and operation can be either eliminated or brought to an extreme minimum. In paragraphs (*h*), (*i*), (*j*), and (*k*) personal errors have received careful treatment to make possible their elimination or reduction to a minimum.

Practically the only thing overlooked seems to be the possibility of using short-wave intercommunication sets, similar to those used by the U. S. Forest Service, for transmitting signals between the instrument man and the sight tender.

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[8] Administrator, Eng. Defense Training Courses, Carnegie Inst. of Technology, Pittsburgh, Pa.

The description of the sequence used in the establishment of the main alinement points AW, AE, AE1, etc., as well as the placing of this line on the various monolith bars, should be treated at greater length. It is hoped that when the author makes his closing statements he will describe in detail the program followed in setting all points by use of the longest foresight available.

C. L. Garner,[9] M. Am. Soc. C. E. (by letter).—An excellent description of the accuracy that can be obtained with modern instruments used in measuring horizontal angles, and for alinement projects of the type described, has been given by Mr. Hough. There is no doubt but what the locations of points by alinements obtained with present-day optics by visual observation over short distances, combined with direct measurements, are much better than can be obtained through methods of triangulation which involve unusual refinement in many steps of the work, including reading of the circle graduations of the instrument. Stated in another way, the method of alinement described is a means of using the theodolite merely as a high-grade sighting device, thus eliminating some of the errors that always exist (even if they are small) in the graduation of theodolite circles and in the lack of perfection in setting up the instrument and the objects to be sighted upon at the various stations where the work is to be accomplished by triangulation. Undoubtedly, also, the method eliminates some of the errors of the personal equation.

On triangulation the difficulty of obtaining acceptable triangle closures involving short lines is due to the fact that, as the distances to the objects observed are shortened, the necessity of accuracy and refinement in the setting of targets and the avoiding of phase, eccentricity, or lack of symmetry becomes greater. All of these have a direct influence on the accuracy of the work as evidenced by the triangle closures. Where they can be held in proportion to the accuracy sought, there is no question as to the capability of the instrument to furnish final results comparable to those which may be obtained through the alinement method. Where the lines are short or even a mile long, these problems become more and more difficult, and consequently the statement that "good triangle closures on extremely short lines are impossible" is true from the practical standpoint.

It is generally recognized that refinements necessary to obtain the highest accuracy where very short lines are involved outweigh the advantages of triangulation except where it is necessary to determine distances across streams or to inaccessible points. Where practicable, direct traverses are preferable.

For some years, the U. S. Coast and Geodetic Survey has employed first-order triangulation and traverses to detect small earth movements over areas of seismic activity. By repeating the observations from time to time as necessary, it is possible to determine progressive or other movements relating to the earthquake problem. Traverse measurements are used in locating the monuments that are closely spaced, and triangulation is employed in the location of monuments that are generally more than a mile apart. Thus it is possible to detect relative movements between consecutive monuments with an accuracy somewhat in proportion to the distance separating them.

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[9] Comdr., and Chf., Div. of Geodesy, U. S. Coast and Geodetic Survey, Dept. of Commerce, Washington, D. C.

In 1939–1940 the U. S. Coast and Geodetic Survey has cooperated with the Navy Department in alinement surveys in connection with its model basin project at Carderock, Md. This work was apparently quite similar to that described by Mr. Hough. On this particular project it was possible to obtain a higher degree of accuracy than ordinarily could be expected because the observations were made under ideal conditions in enclosed buildings of almost constant temperature, something rarely attainable in the field.

Incidentally, it is of interest to note that the theodolite described by Mr. Hough is an American product. A number of these instruments, the original of which was designed and built by the U. S. Coast and Geodetic Survey, have been in successive use since 1928. It has a 9-in. circle graduated to 5″ of arc and it is equipped with two micrometers 180° apart, which give readings of horizontal angle to single seconds. A distinctive feature of this instrument is its nonbinding center so that there is no binding or looseness in the movement of the horizontal circle due to temperature changes. For night observing, the illumination of the cross hairs and micrometer drums is provided through the center axis, the electric energy being supplied by the ordinary 6-volt dry-cell battery. The use of micrometer theodolites reading to one or two seconds of arc is not widespread in the United States. Such instruments must be manufactured on special order. However, during recent years, there has been a growing demand for more precise surveying instruments, and it is believed this demand will soon result in the wider use and adaptation of better surveying methods and improved equipment as necessary therefor.

A most important consideration in triangulation observations is that the instrument must be on a stable foundation, not subject to disturbance by the observer in walking around the instrument. In soft soil it becomes necessary to obviate this by installing a light platform, generally of 1-in. by 12-in. boards, raised off the ground so that the observer's weight is supported at a distance considerably away from the instrument support itself. On precise observations it is also good practice not to change the focus of the telescope during observations. Some freedom of the parts is necessary in order that focusing can be accomplished, and the changing of focal length of the telescope during observations contributes to inaccuracy where short lines of different lengths are used.

Unquestionably the alinement survey principle has its advantages in locating points set for the purpose of studying movement in large concrete monoliths such as that contained in the Tygart Dam described by Mr. Hough. It is a most excellent means of determining small movements which may result from any cause whatever. To accomplish the same result by triangulation becomes more laborious as the scope of such a project is limited.

Floyd W. Hough,[10] M. Am. Soc. C. E. (by letter).—Interested engineers, in discussing this subject, have emphasized the fact that a high order of accuracy on either alinement or triangulation, where short lines are involved, can be obtained only by a careful evaluation of observational errors arising from various sources. Small accidental errors are inherent in any type of direct observational measurement. It is essential that studied methods of operation be used that will of themselves eliminate or make inappreciable all tendency to constant or variable systematic errors, while, at the same time, they are reducing to a minimum the amplitude of the ever-present accidental errors.

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[10] Geodetic Engr., U. S. Coast and Geodetic Survey, Washington, D. C.

As Major Bowie has shown, there is too often a false economy practiced by the heads of engineering organizations in endeavoring to "get by" with the cheaper type of instrument. Even when it is possible to obtain finally satisfactory results from extensive observations with an inferior instrument, an analysis of total cost will show that over a short period of time the difference between initial costs of the inferior and the superior instrument will be entirely absorbed by the more rapid progress of the work with its consequent reduction in cost of labor. It should be borne in mind, too, that with reasonable care the depreciation in value of good surveying instruments is small, their life extending often from twenty to forty years or more.

The writer agrees with the principle inferred by Mr. Heaton that there is no observational advantage gained in reducing the width of slit after a certain reduced width is reached which, in the case of the Tygart Dam work, was believed to be ⅛ in. If a wider slit were used, however, the use of thin paper or ground glass to eliminate possible errors of eccentricity of the bulb and reflector would be required, as Mr. Heaton has stated.

Both Mr. Dell and Professor Stanton suggest that more detail should be given as to the exact sequence followed in establishing the alinement points. The proper sequence of observations is important and the writer is pleased to elaborate on this procedure as follows:

Stations AE1 and AW2 (Fig. 2) were originally fixed as the base alinement monuments from which all other points on the main alinement were derived. (Stations ASE1 and ASW1 were likewise fixed base points for the independent spillway line.) In making the extensions later to AE and AW, the telescope was plunged to determine a preliminary setting of AE and AW approximately on the fixed line AE1–AW2, within an inch or so, by use of the shuffleboard. The theodolite was then moved, for example, to AE1 and the shuffleboard to AW2. With a light on the preliminary mark at AW, a large number of observations were then read on the shuffleboard tape at AW2, using all precautions to minimize observational errors. The mean reading determined the small offset, at AW2. Knowing this offset and the distance between the monuments, the final point at AW was then found by measuring directly on the alinement bar a computed small right-angled offset from the formerly established preliminary point. A similar procedure was used in the determination of the extension point AE.

The "master alinement bar" A13 was set, as described in Table 1 and its supporting text, with the theodolite at the base point AW2, foresighting on base point AE1, then dropping down to the shuffleboard and light at A13. (It will be noted that there was a smaller vertical angle at AW2 than at AE1, and A13 was only one half as far from AW2 as from AE1.) With A13 fixed in position, the theodolite was set up on A13, and A22 was fixed by foresighting on AW2. Likewise, with the theodolite at A13, foresighting on AE1, AE4 was set. Similar procedure was used in locating all other bars on the line.

The spillway line was established by corresponding methods. It is entirely independent from the main line and approximately parallel to it.

All observed points on both lines were located by foresighting on a relatively distant control point, with the exception of one monolith, A34, which was set by plunging the telescope for a short distance with the theodolite at A33 and backsighting on a distant point.

The six transverse intersections were established by use of the shuffleboard on lines connecting triangulation stations. The instrument was set on the nearest station to the intersection and foresighted on the more distant station in each case.

The suggestion has been made that it might have been advantageous to place additional alinement bars in each monolith opposite those of the main line. There are bars on both alinement lines in monoliths 13 and 22, although not directly opposite each other. Additional bars were not placed in the other monoliths as it appeared that their value would scarcely justify their cost, although this may be open to question. It is likely that any appreciable movement in a monolith would be at right angles to the face of the dam, and both bars in the same monolith would necessarily show the same upstream or downstream movement. The monoliths are separated only by narrow expansion joints. Hence, any appreciable displacement of a monolith in azimuth would be accompanied by a similar twisting movement in adjacent monoliths, thus throwing several bars of the main line or spillway line out of position. This would subject the movement to detection by the measurement of successive increases in the offsets of adjacent monoliths. The opposite bars would serve as checks, however, by furnishing on the original alinement a comparison of the computed distance (if the two alinement lines are not exactly parallel as in this case) with the measured distance between bars of the same monolith.

The use of transceivers, or short-wave intercommunication sets, was attempted at the start of the alinement work but was discontinued in favor of the simpler flashlight signals and the international Morse code. However, there is no reason why these or similar transceivers could not be made more dependable and become entirely satisfactory. They were used with some success on other triangulation work of the Pittsburgh District. Similar equipment is used by the U. S. Army as well as by the U. S. Forest Service. It is believed that the U. S. Coast and Geodetic Survey has experimented also with short-wave communication sets to determine their adaptability to triangulation work.

In the measurement of small earth movements in regions of seismic activity, it would seem possible that alinement lines laid down across known fault lines or zones, and extending back to stable ground on each end, would serve as valuable supplements to traverse and triangulation control. They would be adaptable particularly to terrane where the lines are somewhat short for triangulation and too rough to make accurate traverse measurements. Contiguous alinement lines across fault zones should lie at widely varying azimuths up to 90° to allow detection of movement in any direction.

Commander Garner calls attention to the fact that the theodolite used at the Tygart Dam was an American product. The instrument used for the final alinement of the long tunnels of the Colorado River Aqueduct in California was of the same type, designed by the U. S. Coast and Geodetic Survey and built by an American firm. Its accuracy is at least equivalent to, and its adaptability in the field certainly exceeds, that of the finest theodolite made abroad. Although it is true that such instruments must still be made on special order in the United States, the writer believes that appreciation of the economy in using high-grade instruments is growing so rapidly that American manufacturers will soon by justified in producing these and similar instruments in sufficient quantity to enable them to reduce their costs considerably.

Major Bowie makes the following statement: "That knowledge of the degree of stability of a dam structure and of the surrounding area is desirable admits of no doubt." It is desired here to present an actual occurrence which would seem to substantiate this contention.

The failure of the St. Francis Dam in California occurred on March 13, 1927, with its attendant great loss of life and property. During a court investigation of the disaster, the City of Los Angeles stated its belief that an earth movement was responsible for the catastrophe. The writer was asked to present evidence to the court concerning this claim. He made a thorough study of the surveys in the vicinity of the dam at the time of its construction and of those subsequent to the failure, with specific attention to the methods used and the reliability of the resultant data.

It was discovered that certain noticeable and suspicious changes did exist in angles and position, all of a consistent trend, pointing to the probability of earth movement. However, it was also found that the triangulation done during the construction of the dam was of third-order accuracy. The magnitude of the apparent movements was still within the limits of observational errors for this class of control. In view of the latter fact, it was necessary, of course, to go into court and testify that, in the writer's opinion, there was not sufficient proof to warrant a conclusion that there had been any disturbance of the earth's crust contiguous to the dam.

Had a first-order alinement and triangulation control survey been made at the time of the construction of the St. Francis Dam, comparable to that made at the Tygart Dam, based on certain fixed and recognized specifications for that class of work, it would have been possible to prove what changes, if any, had taken place. Due to the lack of adequate survey control, therefore, the city had no satisfactory evidence to support its warranted suspicions of earth movement and the answer as to whether an Act of God was a contributing or even a major cause of the disaster cannot be known. Although such a failure is happily of rare occurrence in modern engineering design, it seems obvious that the ability to study and evaluate, at any time, the conditions of the physical stability of an engineering structure and of the surrounding terrane, amply justifies the expenditure of the small percentage required of the total cost of the structure.

In closing, the writer wishes to record his appreciation for the interest shown and particularly for the contributions and suggestions made by those who have discussed the paper.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 2113

FOUNDATION EXPERIENCES, TENNESSEE VALLEY AUTHORITY

A SYMPOSIUM

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Note.—Published in March, 1940, *Proceedings*.

FOREWORD

When the Tennessee Valley Authority was established by Congress, in 1933, its zone of influence included two completed power developments—Wilson Dam, a federal project, and Hales Bar Dam, a private project. The new authorization extended the development of the valley by the proposed construction of additional dams. An important by-product of this activity—especially to the civil engineering profession—has been the imposing mass of technical data thus made available.

The papers of this Symposium, restricted as they are to foundation experiences, demonstrate the comprehensive nature of the entire project. Three dams are selected as being typical for this purpose—Norris, Guntersville, and Chickamauga. In each paper the author has confined his scope either to geology or to the practical foundation treatment required at the dam site under consideration.

In an effort to avoid the appearance of advocating special commercial interests, the names of manufacturers of equipment described in this Symposium have been omitted. To the same end, it is expected that discussers will confine their comments to matter within the intended scope of each paper.

Personnel.—The projects were built with the Authority's forces. From the time TVA was authorized in 1933 Arthur E. Morgan, M. Am. Soc. C. E., was chief engineer, until he relinquished that position June 17, 1937. At the beginning of the work described in this Symposium, C. A. Bock, M. Am. Soc. C. E. (chief consulting engineer May 1, 1938, to November 30, 1939), was assistant chief engineer; T. B. Parker, M. Am. Soc. C. E., joined the staff in November, 1935, as chief construction engineer, and became chief engineer in May, 1938. He was succeeded as chief construction engineer by A. L. Pauls, M. Am. Soc. C. E.; Edwin C. Eckel, Affiliate Am. Soc. C. E., was chief geologist; and Berlen C. Moneymaker, Assoc. M. Am. Soc. C. E., assistant chief geologist.

On the Norris Dam, Barton M. Jones, M. Am. Soc. C. E., was project construction engineer; C. Douglas Riddle, Assoc. M. Am. Soc. C. E., assistant construction engineer, was succeeded successively by Frederick A. Dale and A. M. Komora, Members, Am. Soc. C. E.; and Ross White, M. Am. Soc. C. E., superintendent of construction, was succeeded by F. C. Schlemmer, who subsequently became construction superintendent of Chickamauga Dam.

At Chickamauga Dam, Lee G. Warren, M. Am. Soc. C. E., was project engineer; James B. Hays, M. Am. Soc. C. E., construction engineer; J. K. Black, Assoc. M. Am. Soc. C. E., assistant construction engineer; and G. E. Murphy, assistant construction superintendent, succeeded by James S. Lewis, Jr., Assoc. M. Am. Soc. C. E.

At Guntersville Dam, Verne Gongwer, M. Am. Soc. C. E., was project engineer; George K. Leonard, M. Am. Soc. C. E., construction engineer, succeeded by F. E. Bell; H. L. Broadfoot, assistant construction engineer; George P. Jessup, construction superintendent, succeeded by A. W. Sherman; and B. S. Philbrick, M. Am. Soc. C. E., assistant construction superintendent.

FOUNDATION TREATMENT AND RESERVOIR RIM TIGHTENING AT NORRIS DAM

By James S. Lewis, Jr.,[1] Assoc. M. Am. Soc. C. E.

Synopsis

The foundation of Norris Dam, in Tennessee, is composed of dolomite, a stratified rock of sedimentary origin. Preliminary investigations disclosed that the formation was characterized by the existence of openings that would permit the passage of water under the structure. The certainty that damaging leaks of large volume would result when the reservoir was filled, if the openings in the foundation were not sealed, made it desirable that a comprehensive program of foundation treatment be included in the plans for the construction of the dam. It was also necessary to treat parts of the reservoir rim.

Under the dam, the treatment was divided into two parts: Shallow, low-pressure grouting of the foundation area to reduce uplift and to consolidate the rock; and deep, high-pressure grouting to form an impermeable curtain under the heel of the structure. On the reservoir rim, the work involved the determination of the location of the parts in need of treatment and the grouting of the parts found to be faulty.

Under the structure, all seams intercepted by the holes for both shallow and deep grouting were carefully washed free of unsound material before grouting. The holes for the shallow grouting were divided into interlocking patterns, whereas those on the line of the curtain were drilled in groups. The effectiveness of the work was improved greatly by the series of operations made necessary by this arrangement.

In the reservoir rim, consolidation rather than replacement of the material in the seams was desired, and washing was not attempted. It was realized that a large volume of material would be required for this work and, in the interest of economy, rock flour was added to the cement grout.

This paper contains a description of the equipment used and of the field practices found to give best results in handling the problems that arose. Cost and progress records are included.

In an effort to avoid the appearance of advocating special commercial interests, the names of manufacturers of the equipment described in this paper have been omitted. To the same end, it is expected that discussers will confine their comments to matter within the intended scope of the paper.

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[1] Constr. Supt., Watts Bar Dam and Steam Plant, TVA, Watts Bar Dam, Tenn.

INTRODUCTION

The construction of Norris Dam by the Tennessee Valley Authority (TVA) was begun in October, 1933, and the project was officially dedicated on March 4, 1936. The maximum height of the dam is 265 ft and the length 1,860 ft. Fig. 1 is a photograph of the completed structure.

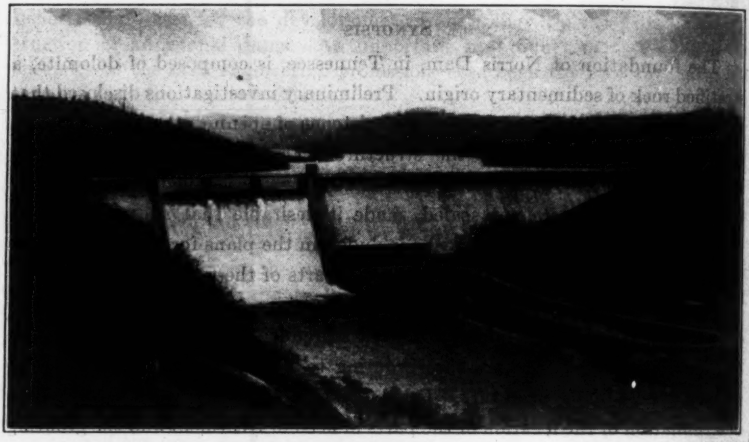


FIG. 1.—NORRIS DAM

The history of the treatment of dam foundations has proved the value of grouting before water is impounded if the nature of the foundation is such as to indicate that excessive leakage may occur. It is expensive and discouraging to attempt to stop foundation leaks that appear after a reservoir has filled, and the TVA determined to spare no reasonable efforts in its attempt to prevent difficulties of this nature at Norris Dam. The foundation of the structure and those parts of the reservoir rim near the abutments were investigated by means of diamond core holes, drilled as part of the preliminary studies made in determining the location of the site. The information obtained in this manner was later supplemented by a geological and a geophysical survey of the site and the region. From the knowledge secured, it was ascertained that the filling of the seams and other openings that existed in the rock would be an undertaking of major proportions.

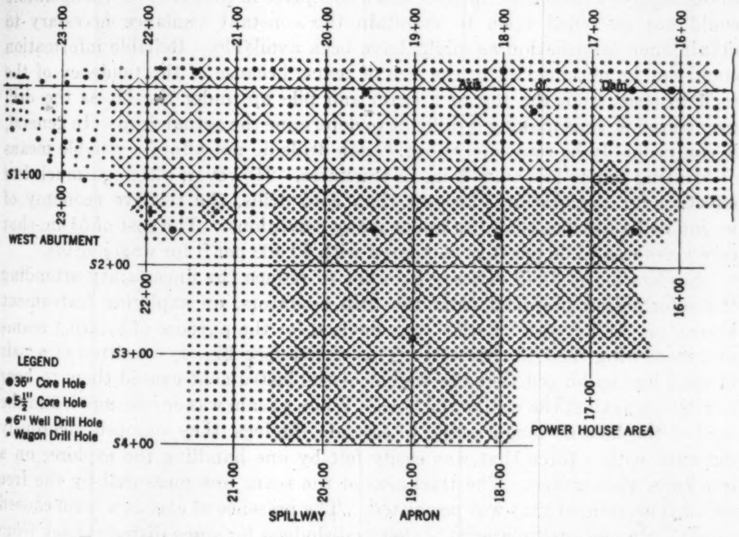
The exceptional care that was exercised to prevent the occurrence of leaks through the foundation and the reservoir rim of the dam has well repaid the effort that was expended. The volume of both drilling and grouting was large, and it was necessary to use a variety of methods to cope with the problems that arose during the prosecution of the work.

GEOLOGY

The geological survey revealed that the rock on which the dam would rest is composed of a hard dolomite banded with numerous seams. Before grouting operations started some of the seams were filled with clay and some were partly open. In addition, the formation is characterized by the existence of large solution channels, weathered joints, and fissures. At the site, the dip is about 5° in a downstream direction, the strike being roughly parallel to the axis of the dam. The rock in the immediate vicinity of the chosen location is known as Rockhouse dolomite, a subdivision of the Knox dolomite that underlies the region. The name Rockhouse evidently alludes to the numerous caves which are found along the bluffs and cliffs bordering the river and which could afford shelter in case of necessity. This brief description of the geology of the region is intended to afford a picture that is typical of the dam foundation and of the reservoir rim. Numerous extensive seams, caverns, and solution channels would have to be sealed in order to insure against serious leakage.

Early Foundation Investigations

Closure of the first cofferdam required some grouting in order to stop leaks through seams into the enclosure. In this preliminary work, it was learned that both open and clay-filled seams were so numerous and extensive that only a systematic plan of treatment would succeed in tightening the foundation effectively. Accordingly, after some study, a plan was evolved whereby the entire foundation was to be drilled on a grid of wagon and core-drill holes at the intersections of lines 10 ft apart, running normal to, and parallel with, the axis of the dam. Figs. 2 and 3 show the manner in which this grid was divided into interlocking patterns, designated as *A* and *B*, with a 5.5-in. hole at the center of those patterns under the gravity section. The plan also called



23+00
22+00
21+00
20+00
19+00
18+00
17+00
16+00
Axis of Dam
S1+00
23+00
WEST ABUTMENT
S2+00
22+00
16+00
S3+00
LEGEND
● 36" Core Hole
● 5½" Core Hole
● 6" Well Drill Hole
• Wagon Drill Hole
S4+00
17+00
POWER HOUSE AREA
21+00
20+00
19+00
18+00
SPILLWAY
APRON

FIG. 2.—PART PLAN OF SHALLOW GROUT HOLES AND INVESTIGATION HOLES

for the exploration of the holes in order to locate all seams, and the thorough washing of these seams to remove clay and loose material. As soon as the foundation rock was exposed in the first cofferdam, the drilling of exploratory holes was started with wagon drills and shot core drills.

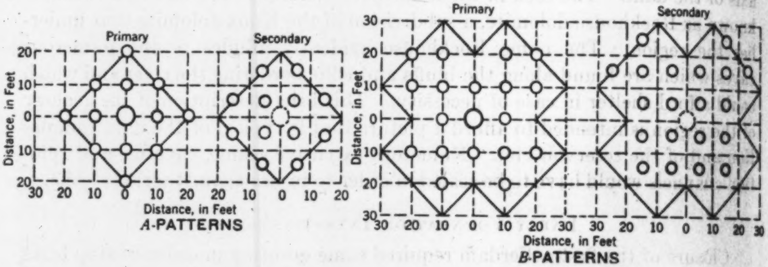


FIG. 3.—TYPICAL PATTERN LAYOUT AND DRILLING DIAGRAM

Locating Seams with Wagon Drills.—It was hoped that the operators of the wagon drills could determine the occurrence and thickness of seams, satisfactorily, and report them to a recorder. However, it was sometimes difficult to distinguish between an opening and a soft place in the rock. Drill operators could not be relied upon to maintain the constant vigilance necessary to obtain such information as might have been available. Reliable information could not be obtained as to the size of seams because of the tendency of the rock to spall from the top of a seam and pile up under the bit as the drill penetrated it, thus making it difficult to determine the actual drop. In general, it can probably be said safely that wagon drills do not afford a reliable means of locating seams less than 3 in. in thickness. However, although much less effective for exploration purposes than core drills, the relative economy of wagon drills justifies their use under many conditions. The cost of 5½-in. shot core holes was $3.66 per ft as compared with $0.408 per ft for wagon drills.

Exploration of Drill Holes.—In order to reduce the uncertainty attending the information obtained from the drill operators, an exploring instrument known on the job as a "Feeler" was devised for the purpose of locating seams and measuring their thicknesses. This device (see Fig. 4) consisted of a pair of steel legs so hinged that the weight of the instrument caused them to bear outwardly against the wall of the hole. When a seam was encountered, and the wall of the hole no longer acted to restrain the legs, they snapped vigorously outward with a force that was easily felt by one handling the explorer on a line from the surface. The thickness of the seam was measured by the free vertical movement that was permitted. The presence of clay in a seam caused no difficulty, as such material is always dislodged for some distance back from the wall of the hole. The device was suspended on a marked ½-in. wire rope and was equipped with a latch that released the legs automatically when the bottom of the hole was reached. The hole was explored by pulling the "Feeler" from the bottom upward. The method was found to be satisfactory in wagon-drill holes and, by increasing the length of the legs, it was used with even better results in vertical 5½-in. core holes. It is not desired to imply that the method has no shortcomings. Its effectiveness depends to a considerable extent upon the experience of the operator and his skill in interpreting the indications that he obtains. In rough-walled holes, such as those drilled with wagon and churn drills, the effectiveness is not so great as in the comparatively smooth core holes.

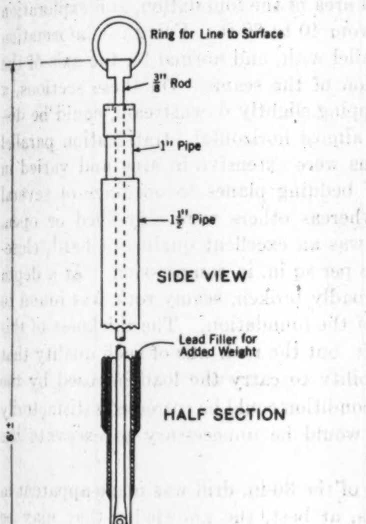

Ring for Line to Surface
3/8" Rod
1" Pipe
1½" Pipe
SIDE VIEW
Lead Filler for Added Weight
HALF SECTION
Links
Fixed Hinge
Hinged Legs
Latch to Hold Legs in While Lowering. Contact With Bottom Automatically Releases Latch

FIG. 4.—MECHANICAL HOLE EXPLORER

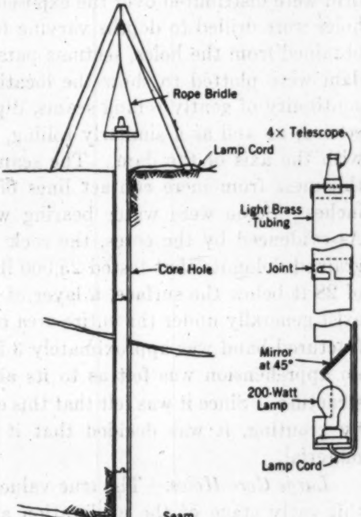

Rope Bridle
4× Telescope
Lamp Cord
Light Brass Tubing
Core Hole
Joint
Mirror at 45°
200-Watt Lamp
Lamp Cord
Seam
SECTION THROUGH HOLE

FIG. 5.—PERISCOPE FOR INSPECTING CORE HOLES

Periscope.—A periscope (see Fig. 5) was designed for checking the results of the exploration of the 5½-in. core holes by the "Feeler." The periscope was made of brass tubing in sections 10 ft long, graduated in feet to facilitate reading depths. It was used at a maximum depth of 75 ft; but, in the form described, it was not suitable for use under water or in crooked holes. If the length of section was reduced to 5 ft it could be handled more easily by the observer as, with long sections, he must either be lowered in a sling or must climb down a ladder while keeping his eye at the telescope. This periscope was used successfully to test the effectiveness of the explorer, after which it was abandoned in favor of the simpler and faster "Feeler."

Foundation Studies.—During the early stages of the excavation, it was considered desirable to secure as much information as possible so that it could be used in formulating a plan to be applied flexibly to the foundation treatment of the entire project. For this purpose five 5½-in. shot drills and one 36-in. shot drill were distributed over the exposed area of the foundation, and exploration holes were drilled to depths varying from 40 to 80 ft. Using the information obtained from the holes, sections parallel with, and normal to, the axis of the dam were plotted to show the location of the seams. On these sections, a continuity of gently rolling seams, dipping slightly downstream, could be discerned, as well as a similarly rolling, almost horizontal, stratification parallel with the axis of the dam. The seams were extensive in area and varied in thickness from mere contact lines of bedding planes to openings of several inches. Some were water bearing whereas others were clay-filled or open. As evidenced by the cores, the rock was an excellent quality of hard, close-grained dolomite that tested 25,000 lb per sq in. in compression. At a depth of 28 ft below the surface, a layer of badly broken, seamy rock was found to exist generally under the entire area of the foundation. The thickness of this fractured band was approximately 3 ft, but the rock was of such quality that no apprehension was felt as to its ability to carry the load imposed by the structure. Since it was felt that this condition could be corrected satisfactorily by grouting, it was decided that it would be unnecessary to excavate the material.

Large Core Holes.—The true value of the 36-in. drill was made apparent in this early stage of the exploration as, at best, the knowledge that may be obtained from small core holes is limited. Grinding and blocking of the core within the core barrel act to destroy the original appearance of the rock and it is often difficult to judge whether core loss results from attrition or from a seam. This fault may be reduced to some extent by forcing the drillers to pull the cores frequently. Actually, it is the operator of the drill who must interpret, by the action of his machine, the evidence as to subsurface conditions signaled through the tools. There is no wish to detract from the value of borings of small size but rather to emphasize the value of large borings. The value of an investigation made by means of small holes is greatly increased when it is supplemented by holes large enough to permit the entry of a man. In the large holes, the rock may be studied in its original, undisturbed state to an extent that is not possible in shafts that have been excavated with explosives, wedges, and percussion drills. The large cores do not suffer from grinding and blocking as they do not rotate within the drill bit.

The greatest limitation upon this work is imposed by the ground water, and the depth at which the inflow becomes so great that the driller is unable to work in the bottom of the hole to remove core usually fixes the depth to which the hole may be drilled without sealing the rock in advance. A crew consisted of one driller with two helpers, and the average drilling rate was 0.33 ft per hr

Holes reaching a depth of 57 ft were drilled for exploration purposes by first drilling four wagon-drill holes on the corners of a 10-ft square and grouting these holes before the core hole was started in the center of the square. Considerably greater depth could be reached in any case by advance grouting of the region to be penetrated.

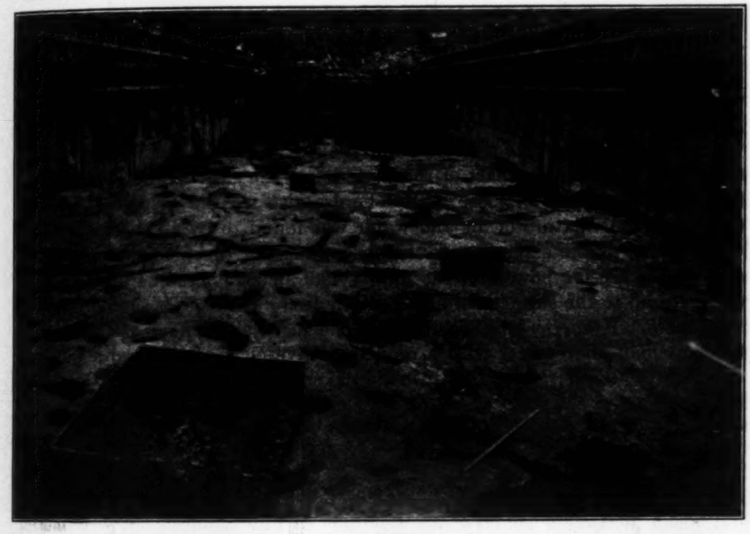


Fig. 6.—Typical Foundation

In determining the general plan of treatment to be followed, the knowledge obtained from drill records, exploration records, inspection of 36-in. holes, and observations of the structure of exposed rock was correlated to form the basis of the final plan. As the result of this study, it was decided to remove the rock under that part of the gravity section lying in the river bed down to a clay-filled seam that had been found to underlie the rock surface at a depth of approximately 12 ft. Under the spillway apron and power house the excavation was to be extended only sufficiently deep to meet construction requirements or to remove weathered and unsound material. The surface of all of the rock exposed at seams or bedding planes was found to be dipping gently downstream and was marked by frequent domes and hollows that would afford an excellent mechanical bond to resist sliding. The great frequency of these irregularities is shown clearly in Fig. 6.

Drilling.—Referring to Figs. 2 and 3 showing the layout of the holes and the system of interlocking patterns that was chosen for the shallow grouting, the plan called for the drilling, washing, and grouting of the *A*-patterns in an area before the *B*-pattern holes were drilled. Areas were ordinarily not less than 100 ft square. The grouping greatly facilitated the washing of unsound material from the seams and the later replacement of this material with grout, as the manner in which the *B*-patterns surrounded the *A*-patterns permitted a cross flow between holes that covered practically every square foot of the area. Grouping also allowed a large number of holes to be drilled simultaneously, thus contributing toward the improved progress of construction at a stage when delays could prove very costly.

WASHING SEAMS FOR SHALLOW GROUTING

The preliminary grouting to seal the first cofferdam revealed that the holes in any group were usually interconnected by seams and that it was frequently possible to wash away large quantities of clay and unsound material by forcing water and air into different holes of the group. The water emerging from the other holes would bring part of the material to the surface and it is probable that large quantities of loose material were also washed to distant areas in the seams. A mixture of air and water was found to be more effective than water alone, as the expansion of the air produced a turbulent action that increased the erosive powers of the fluid. The air also acted as a lift to bring the material to the surface through other holes.

Flow-Reversing Device.—To improve the dislodging effect of the washing fluid, a small air receiver with a reducing valve on the inlet and a four-way valve on the discharge, as shown in Fig. 7, was used for reversing the flow of the air. In washing a pattern, it was customary to connect a supply of water to one hole and to connect air from the receiver to holes on opposite sides of the water connection. By operating the four-way valve manually, at frequent intervals, the water in the seams was maintained in a turbulent state that was

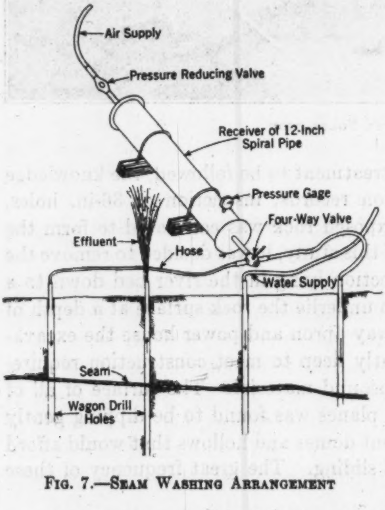

Air Supply
Pressure Reducing Valve
Receiver of 12-Inch Spiral Pipe
Pressure Gage
Four-Way Valve
Effluent
Hose
Water Supply
Seam
Wagon Drill Holes

FIG. 7.—SEAM WASHING ARRANGEMENT

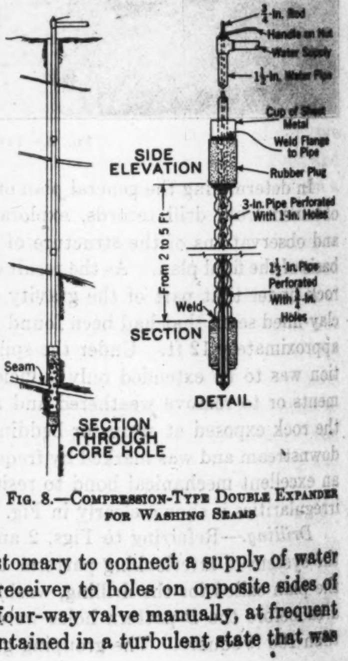

¾-In. Rod
Handle on Nut
Water Supply
1½-In. Water Pipe
Cup of Sheet Metal
Weld Flange to Pipe
SIDE ELEVATION
Rubber Plug
3-In. Pipe Perforated With 1-In. Holes
From 2 to 5 Ft
1½-In. Pipe Perforated With ¾-In. Holes
Weld
SECTION
Seam
DETAIL
SECTION THROUGH CORE HOLE

FIG. 8.—COMPRESSION-TYPE DOUBLE EXPANDER FOR WASHING SEAMS

very effective in dislodging clay and decomposed rock. An air motor mounted on the receiver to operate the four-way valve at regular intervals would improve the efficiency of this equipment. When the washing operation was begun, most of the holes were capped and, as the flow from the open holes became clear, the caps were moved about to change the path of flow of the fluid. In this manner each pattern was washed until the clarification of the effluent water indicated a reasonable freedom from clay and other undesirable material. The methods described were used under the spillway apron and power house.

Washing Individual Seams.—Under the gravity section, where it was desirable to secure an exceptionally tight seal, each seam was isolated and washed individually by means of a double expander (see Fig. 8) which was inserted in the 5½-in. core hole at the center of the pattern after exploration of the hole had disclosed the location of the seams. The expander was placed so that the rubbers were an equal distance on each side of the lowest seam. The expanding nut was then tightened and water and air forced in until all possible interconnection with the wagon-drill holes in the group had been established. As soon as reasonably clear water flowed from a hole, it was capped in order to force a passage to other holes. Frequently, holes that offered great resistance at the beginning of washing operations would open and take water quite freely after pressure had been applied for several hours. The washing was always started at the lowest seam in a hole, and progressed upward. Normally, the rubbers on the double expander were spaced 24 in. apart, but, when a closely seamed zone that exceeded this spacing in thickness was encountered, the spacing was increased. At the same time that the seams underlying a pattern were being washed individually through the core hole, the air receiver used for producing turbulent flow was connected to the wagon-drill holes. At intervals while washing, and to complete the washing operation, every hole was blown clear of accumulated muck with an air jet. The washing was usually timed so as just to precede the grouting. The length of time required for washing a pattern varied between 6 and 24 hr and was largely determined by the judgment of the inspector. Ordinarily, when washing was started, the overflow water from interconnected holes was very muddy, containing small lumps of clay and weathered rock. After some time, the water gradually would become clear, although it was considered neither practical nor desirable to wash until the discoloration had disappeared entirely.

Pressures of air and water for washing shallow grout holes were limited to 30 lb per sq in. and it was necessary at times to reduce this pressure in order to avoid lifting the rock. The safe pressures for washing and grouting were determined by learning the depth to the uppermost seam. The weight of the rock over this seam was then computed. Ordinarily, the theoretical pressure could be exceeded as it was safe to assume that pressure would not exist under the entire area and that the surrounding rock would offer restraint through slab action.

Upheaval Gages.—Upheaval gages, to give an indication when the safe pressure was being exceeded, were placed at intervals over the area of the foundation. These gages (described subsequently) were observed during both washing and grouting operations and frequently served as a warning that the safe pressure was being exceeded.

SHALLOW GROUTING

In general, the system that has been described was used in preparing for the shallow grouting of the foundation area, but it became desirable at times to modify these methods to adapt them to special conditions. Under the spillway apron and power house, where a number of small seams were known to exist near the surface, the *A*- and *B*-patterns were first drilled and grouted to a depth of 20 ft. Then a secondary system of 40-ft holes, evenly spaced between those first drilled, was superimposed upon the original patterns and the same sequence of operations was repeated. In washing and grouting the 40-ft holes, it was not unusual to find a slight connection with nearby 20-ft holes that had been previously grouted to refusal. It was assumed that subsequent shrinkage of the grout deposited in the first operation had opened the seams slightly. Occasionally, throughout the duration of the work, it was found that open passages existed in territory that had been grouted. Besides shrinkage of the grout in the seams, this was attributed to the fact that, when washing in an area which had been previously tightened by grouting, it was possible to build up higher pressures because of the more limited and restricted passageways that existed. The resulting velocity of the wash water was probably considerably higher than in the first operation, and it was safe to assume that unsound material which had previously resisted the lower velocities of the first washing was scoured out. New passages were thus opened. To a lesser extent, this same progressively restricting effect was utilized by adopting the pattern system. The grouting of the *A*-patterns effected some consolidation of the ungrouted *B*-pattern areas, and the greater resistance to flow that resulted when the *B*-patterns were washed increased the velocity and improved the scouring effect.

The pattern system of grouting was followed, in general, over most of the foundation area. Only under the spillway apron and in a small area in the east abutment was the primary pattern system followed by a secondary system extending to a greater depth. Under the gravity section, grouting was always started from the core hole at the center of the pattern and the wagon-drill holes were capped as overflow occurred. After refusal had occurred on the core hole, the wagon-drill holes were grouted individually and ordinarily required very little grout. In general, the area under the spillway apron and power house was grouted to a depth of 40 ft, whereas the area under the gravity section, where the excavation was extended 12 ft deeper, was grouted to a depth of 30 ft.

In order to meet special conditions more adequately or more economically, it was necessary, occasionally, to vary the plans outlined herein. In a relatively small area under the west side of the spillway apron, the plan was altered to expedite the work during an important stage of construction. The alteration simply consisted in dividing the area into three parts that were drilled and grouted separately, the holes being placed at the intersections of lines forming a 10-ft grid.

The west abutment was composed of a number of narrow ledges underlain by seams, and it appeared desirable to discontinue the pattern system and to treat each seam separately. In the east abutment (not shown in Fig. 2), the ledges were much wider and the pattern system was modified to obtain the best results on each one.

Anchorage of Spillway Apron.—In each of the wagon-drill holes in the area of the spillway apron, a bar of 1¼-in. steel was inserted to anchor the concrete apron slab against uplift. When early and wide interconnection developed in a pattern, filling other holes with grout that might harden and prevent the insertion of the rods, the difficulty was overcome by placing tees with plug cocks on the side outlets on all nipples. When grout flowed from a connected hole, the rod was inserted through the tee as soon as the density of the overflowing fluid appeared to equal that of the mixture being injected. The cock on the side outlet was opened until oakum could be calked around the rod at

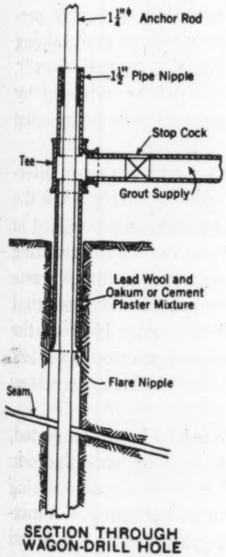

1¼"ϕ Anchor Rod
1½" Pipe Nipple
Stop Cock
Tee
Grout Supply
Lead Wool and Oakum or Cement Plaster Mixture
Flare Nipple
Seam
SECTION THROUGH WAGON-DRILL HOLE

Fig. 9.—Arrangement for Grouting Seams and Anchor Rod Simultaneously

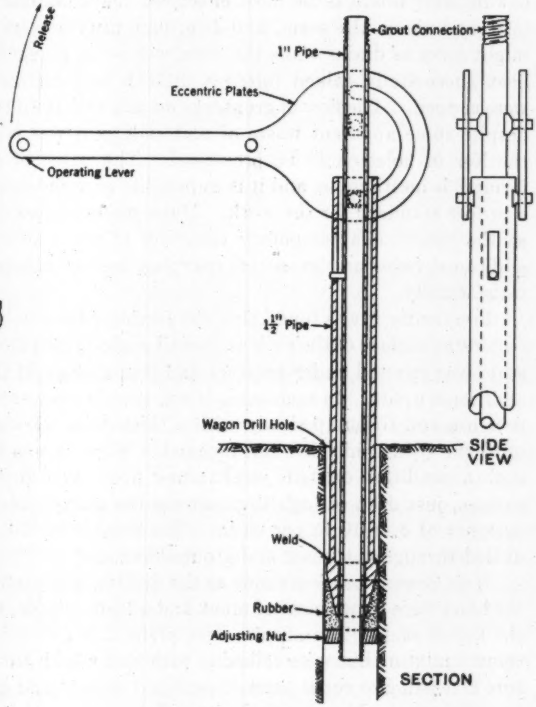

Release
Grout Connection
1" Pipe
Eccentric Plates
Operating Lever
1½" Pipe
Wagon Drill Hole
SIDE VIEW
Weld
Rubber
Adjusting Nut
SECTION

Fig. 10.—Expander for Small Drill Holes

the top of the tee, after which it was closed to permit pressure to build up. Fig. 9 shows the details of the arrangement. When refusal on the original hole had occurred, each hole in the group was grouted through the side outlet of the tee.

Surface Leaks.—When the washing indicated that leakage of grout could be expected to occur where seams intersected the surface, these seams were calked in advance with oakum and lead wool, driven in with a blunt-edged calking tool and backed up with wooden wedges or a dry mortar of quick-setting cement. Where a seam appeared at the base of a vertical face, a rough form was sometimes built a short distance from the face and filled with concrete through which pipes discharged any water that might be flowing from the seam. The pipes were later capped when grout flowed from them. When it was necessary to calk a seam from which grout was flowing, lead wool and oakum were found to be most effective. Jackhammer holes were first drilled into, and along, the seam, and 1-in. pipe nipples were calked in so that they might serve as drains while the seam was being plugged. When the seam had been successfully calked (often a difficult and discouraging task), the pipes were capped. The flow of grout was maintained at all times during the calking, despite some apparent waste of material, as it was only in this manner that the loss of holes could be prevented. The value of grouting, properly performed, is inestimable, and it is impossible to avoid some waste in maintaining a proper standard for the work. Holes made useless (usually termed "lost"), as the result of a temporary cessation of grout flow, must be replaced by additional holes and even this may not insure that the area will be grouted satisfactorily.

Frequently it was found that the feather edge resulting where a seam intersected the surface of the rock at a small angle caused trouble by lifting when the seam was grouted under pressure and that calking of the leak only resulted in additional uplift. In such cases, it was usually necessary to reduce the grouting pressure and to pump slowly with a thick mix, allowing some grout to waste until the seam had filled and plugged. When it was learned in advance that such a condition existed, jackhammer holes were drilled vertically from the surface, just deep enough to penetrate the seam, and it was grouted at a low pressure of 5 or 10 lb per sq in. The deep holes for the patterns were then drilled through this seam and grouted as usual.

Hole Connections.—As soon as the drilling of a pattern had been completed, the holes were blown clear of muck and a 1½-in. nipple, 18 in. long, was placed in the top of each. The nipples were preferably grouted in with a quick-setting cement mixture but were calked in with lead wool if time was pressing. A mixture consisting of equal parts of portland cement and gaging plaster was found to set very quickly and to have sufficient strength to hold the pipe nipples firmly. A quick-working, simple expander was developed after the shallow grouting was completed, and it is certain that economy could have been realized from the use of this device to replace the nipples. The expander is shown in Fig. 10 and its effectiveness has since been tested in the field at Chickamauga Dam near Chattanooga, Tenn.

Equipment.—For mixing the grout, two-compartment, open-top, mechanically agitated mixers that discharged by gravity into the suction of a 7 by 5 by 10, air-driven, duplex, reciprocating pump, were found to be very satisfactory. A corner-type, air-driven drill motor, mounted on a bracket at the end of the mixing tank, was used to drive the agitating blades. The pump and mixer were mounted together as a portable unit that was easily moved to any part of the job. The mixer was divided into two compartments so that an uninterrupted flow to the pump might be obtained by mixing in one compartment while discharging the other. A valve in the branch of the pump suction from each compartment permitted the pump operator to maintain a continuous flow. Fig. 11 shows the arrangement of the equipment in a unit.

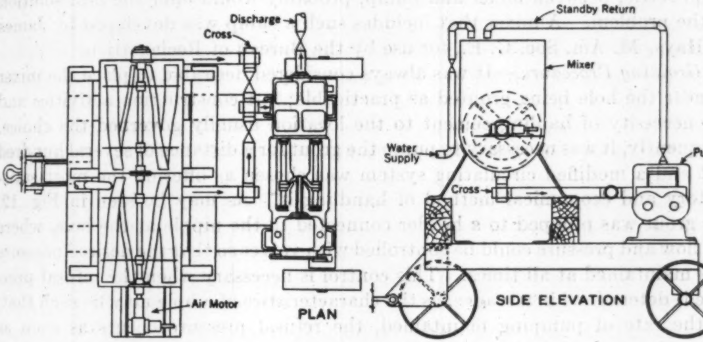


FIG. 11.—PORTABLE GROUT MIXER

The maintenance of the mixers was not an important item, as wear of the shaft and the stuffing boxes where the shaft passed through the end walls and partition was not excessive if reasonable care was exercised in keeping the glands packed. On the pumps, the demand for the repair and replacement of valves, piston packing, and cylinder liners was more or less constant. The fluid pistons were of cast iron with rubber packing, and the cylinders contained removable steel liners. The rate of wear of the rubber piston packing against the steel liners was found to depend largely upon the pressure at which the pump was operating and the wear was approximately equal on the steel and on the rubber. The life of the cast-iron valve seats was also found to depend to a great extent upon the pumping pressure. The seat rings were removed in the field and refaced in the shop. Valve disks of medium rubber were found to be far superior to the fiber disks furnished as original equipment because their greater ability to conform to slight irregularities in the seat made them more effective in preventing leakage. The abrasive action of grout is so great that, once a small leak has started, only a short time is required for considerable damage to result. Piston packing and valves were usually replaced in the field, although it was necessary to have cylinder liners removed and replaced in the machine shop. Four complete grouting units were kept in service, and one spare fluid end for the pumps was always held in good repair to facilitate replacements.

The greatest source of trouble with regard to wear on the pumps was found to lie in the steel particles that were contained in the cement. These particles blocked valves and scored cylinders and, in general, increased the wear on the pumps appreciably. At various times during the grouting operations, screens were installed to remove foreign material from the fluid grout, but they proved such a constant source of trouble and delay that it became desirable to abandon their use. Dry screening of the cement through a screen small enough to remove the particles would have been an expensive operation, and subsequent experience indicated that an agitated sump, into which the foreign material could settle, between mixer and pump, probably would offer the best solution to the problem. A mixer that includes such a sump was developed by James B. Hays, M. Am. Soc. C. E., for use by the Bureau of Reclamation.

Grouting Procedure.—It was always considered desirable to place the mixer as near the hole being grouted as practicable; but construction activities and the necessity of hauling cement to the location usually governed the choice. Frequently, it was necessary to pump the grout for a distance of several hundred feet and a modified circulating system was chosen as offering the most satisfactory and economical method of handling it. As may be seen in Fig. 12, the grout was pumped to a header connected to the nipple at the hole, where the flow and pressure could be controlled with valves so that the desired pressure was maintained at all times. This control is necessary when the refusal pressure is determined in advance, as the characteristics of a hole may be such that, at the rate of pumping maintained, the refusal pressure occurs as soon as grouting is started, although a considerable quantity of grout may be taken by the hole before it closes. By keeping the grout circulating back to the mixer at a fairly high velocity, the tendency of the cement to deposit and form obstructions at fittings is reduced. The velocity that must be maintained in

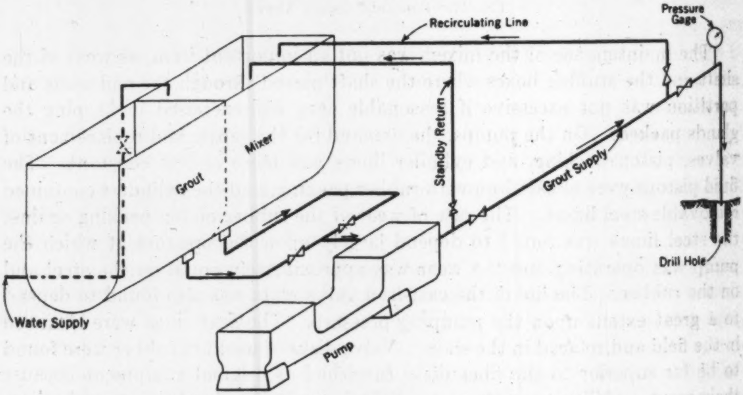


FIG. 12.—CIRCULATING ARRANGEMENT FOR GROUTING

order to keep the lines open depends to a great extent upon the prevailing temperature. The modified circulating system also possesses advantages when gradual refusal is occurring as it gives flexibility in controlling the pressure, regardless of the rate at which the hole is taking grout.

The value of a pump for handling grout, as compared with a compressed air chamber, is great. The pump affords additional mixing of the fluid and makes possible a flexibility of control and continuity of flow that is not obtainable with the pneumatic machine. However, the pneumatic machine is superior for handling grout containing sand or sawdust or the other coarse materials which are sometimes injected into large openings.

TUNNELS TO SEAL LARGE SEAMS IN ABUTMENTS

In addition to the grouting of the seams in the abutments, and preceding it, other measures were taken to reduce percolation to a minimum. When the size of the seams indicated the necessity for extra precautions, tunnels were driven parallel to the axis, following the seams back into the abutments. On one level in the east abutment, five tunnels, spaced about 20 ft apart and varying in length from 100 ft to 250 ft, were driven into a seam until it had closed sufficiently so that it was certain that it could be sealed economically with grout. On other levels, not more than one tunnel was driven, with the exception of some very short ones in the west abutment. The longest tunnel was driven under the core wall that extended eastward through the rolled earth fill at the east end of the gravity section. This tunnel was 671 ft long and followed a very large clay-filled seam that started at the dam at El. 965. The seam varied in thickness from a mere contact plane to 40 in., averaging about 18 in., and the clay varied from compact to very loose, containing some openings. It was not unusual to find stretches of 20 to 40 ft where there was no evidence of the seam other than a discolored band in the rock. Such contact areas necessarily exist in any seam to support the weight of the overlying material.

Filling Tunnels.—To fill the tunnels in the abutments, 36-in. core holes were drilled from the surface and concrete was dumped into the holes in 3-cu-yd batches. The long tunnel under the core wall was filled through an 8-in. pipe from the surface, the concrete being forced in with compressed air.

In order to permit the injection of grout into any space resulting from the shrinkage of the concrete in the tunnels, and also to act as an air vent while placing the concrete, two 2-in. pipe headers were run for the full length of each tunnel and 1-in. pipes connected to the headers were extended upward into jackhammer holes, drilled 12 in. vertically into the roof of the tunnel. The 1-in. pipes were left open and were socketed in this manner in an effort to prevent them from filling with mortar from the concrete. High points in the tunnel roof were chosen as the location for these 1-in. branches. In addition to the pipes inserted in holes as described, others were run back into the seam that the tunnel had followed. These were spaced about 10 ft apart and were installed for the purpose of grouting the seam between the tunnels. They were connected to the same header and dry mortar was packed between and around them where they entered the seam. The arrangement of the grout pipes in one of the tunnels is shown in Fig. 13. After filling the tunnels with concrete, grouting was always deferred as long as possible in order to allow time for shrinkage. After the tunnels had been filled with concrete and grouted, the areas lying between parallel tunnels were drilled from above in the usual manner, and any remaining open spaces were washed and grouted through the holes. Eight tunnels were driven into the east abutment, not including the long one under the core wall, and four were driven into the west abutment.

FIG. 13.—VIEW IN TUNNEL SHOWING GROUT PIPES AND LARGE SEAM

SEALING CONTRACTION SPACES IN ABUTMENTS

In the vertical or steeply sloping rock faces of the abutments, a water stop of 16-gage copper, 14 in. wide, was embedded 3 ft inside of the upstream face of the dam. Jackhammer holes 10 in. deep were drilled on close centers, and the web between them was broached out to form a groove in the rock into which the copper strip was sealed with a stiff mortar. Half the width of the copper strip was allowed to project out of the groove to be embedded in the concrete.

In addition to the water stops that have been described, grout outlets for filling any space resulting from contraction of the concrete were installed on the vertical faces of the rock. The outlets consisted of matched flanges, so arranged that one flange was anchored in the rock face while the other was held by the concrete. It was intended that shrinkage of the mass concrete should separate the flanges, permitting the entrance of grout to the resulting space between rock and concrete. The flanges were connected to pipes running to the gallery in the dam.

INVESTIGATIONS FOR DEEP FOUNDATION TREATMENT

When the shallow grouting was completed, the concreting of the structure had progressed sufficiently so that the deep drilling for the curtain grouting from within the gallery could be started. The section through the spillway in Fig. 14 shows the location of these holes, which varied in depth from 60 to 200 ft, depending upon the conditions encountered. It may be noted that few of the holes extended to the maximum depth. For this curtain grouting, 5.5-in. shot core holes, on the upstream side of the gallery and sloping upstream at an angle of 10° with the vertical, were drilled on 10-ft centers for the length of the dam. However, before drilling the grout holes, vertical upheaval gage holes, spaced 60 ft apart and driven to a depth 10 ft in excess of that tentatively chosen for the grout holes, were drilled. The purpose of drilling these holes in advance was primarily to obtain accurate and detailed information concerning the location of seams. By combining the logs of the drilling and the exploration records of the holes, it was possible, before drilling for grouting was started, to plot a very useful section through the rock underlying the dam, showing the location, thickness, and general characteristics of all the major seams. When developed for the full length of the dam, this section was used as a basis for planning the original program of curtain grouting. As additional

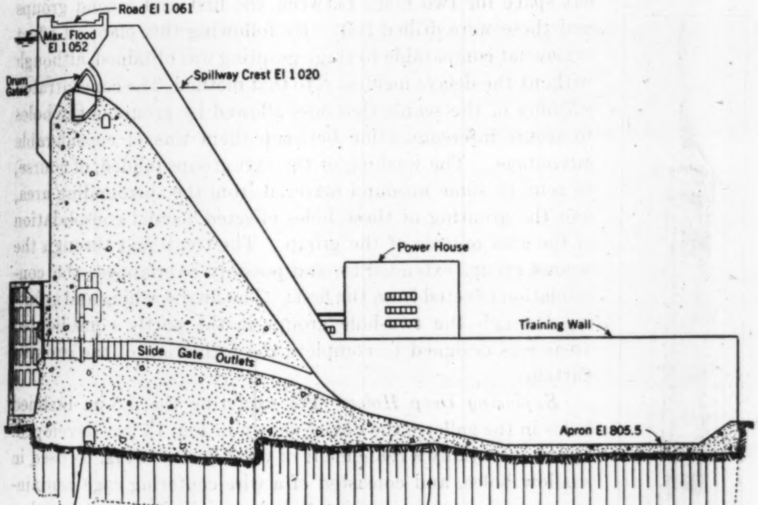

Road El 1 061
Max. Flood El 1 052
Drum Gates
Spillway Crest El 1 020
Power House
Training Wall
Slide Gate Outlets
Apron El 805.5
Boundary of Grout Zone
Shallow Grout Holes Depth 20 Ft to 40 Ft
Curtain Grout Holes Depth 60 Ft to 200 Ft

FIG. 14.—SECTION THROUGH SPILLWAY AND SLIDE GATE OUTLETS SHOWING FOUNDATION GROUT HOLES

information was secured by drilling the grout holes, the program was modified to meet new conditions.

Grouping of Holes.—The work of drilling and grouting the holes 10 ft apart in the galleries was divided into three parts. First, groups of three holes on 100-ft centers between groups were drilled, washed, and grouted. Then groups of three holes half-way between the first groups were treated similarly. This left space for two holes between the first and second groups and these were drilled last. By following this plan, an effect somewhat comparable to stage grouting was obtained, although without the delays incidental to that method. The concentrated washing of the seams that was allowed by grouping the holes to secure interconnection between them was of considerable advantage. The washing of the first groups tended, of course, to remove some unsound material from the surrounding area, and the grouting of these holes effected partial consolidation of the area outside of the group. The treatment through the second groups extended to, and possibly overlapped, the consolidation effected from the first. The final washing and grouting through the two-hole groups in the partly consolidated areas was designed to complete the formation of the grouted curtain.

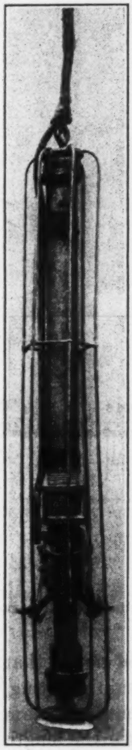


FIG. 15.—ELECTRICAL EXPLORER

Exploring Deep Holes.—For exploring the 5.5-in. inclined holes in the gallery, the device shown in Fig. 15 was developed. It was essentially a refinement of the "Feeler" (Fig. 4) used in shallow holes, and consisted of a wire centering cage containing a pair of spring-actuated legs so adjusted that an electrical circuit was closed, flashing a light for the observer, when either leg moved outside of the travel limits fixed by a hole of normal size. The cage was suspended on a graduated ⅛-in. bronze cable with an insulated single conductor taped parallel and close to it. The bronze cable served as a conductor also. A latch held the legs in the closed position until the bottom of the hole was reached, when they were released automatically and were free to open into any seam encountered as the device was pulled upward. Each hole was explored twice to reduce the risk of missing seams. The device was very effective for locating seams, and the ease of handling afforded by its lesser weight, plus the fact that it was probably superior at the greater depths, gave it a marked advantage over the original explorer.

WASHING THROUGH DEEP HOLES

Following the location of the seams in any group of holes with the exploring device, each seam was washed individually by means of the pneumatic expander shown in Fig. 16. This expander consisted of two sleeves of heavy gum rubber, separated by a piece of perforated pipe, and was suspended from the surface on a marked ¼-in. steel cable, with a parallel hose carrying the washing fluid. A separate ¼-in. hose supplied air to expand the rubber sleeves, and both hoses were lashed to the cable at about 10-ft intervals. By centering the perforated pipe on a seam and expanding the sleeves, the entire supply of washing fluid could be directed into the one seam. This pneumatic type of expander possesses decided advantages where the head-room is limited, as in the grouting galleries, or when the holes are very deep. Where the head-room is limited, the pipe that supports the mechanical type must be cut in short sections, making it awkward to handle; and in deep holes, the increased weight adds to the difficulties. A minor detail is the cup that is mounted on top to catch fragments of rock that fall from seams and shattered zones. If not caught, these fragments will wedge in the space between the expander and the wall of the hole and occasionally result in the loss of the equipment. Sufficient air pressure is required to expand the rubber against the total head of the washing water, at the depth of the seam being washed, if leakage is to be prevented.

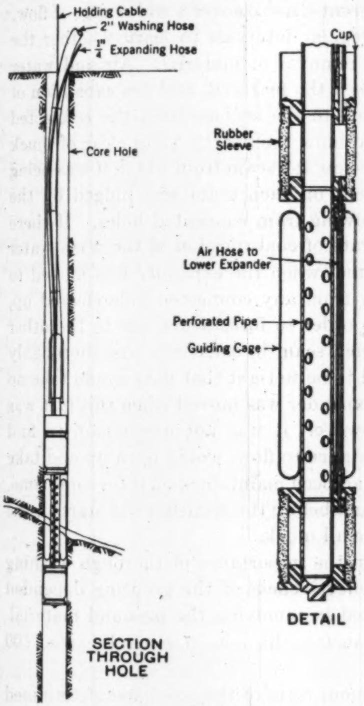

Holding Cable
2" Washing Hose
¼" Expanding Hose
Core Hole
Cup
Rubber Sleeve
Air Hose to Lower Expander
Perforated Pipe
Guiding Cage
DETAIL
SECTION THROUGH HOLE

FIG. 16.—PNEUMATIC EXPANDER

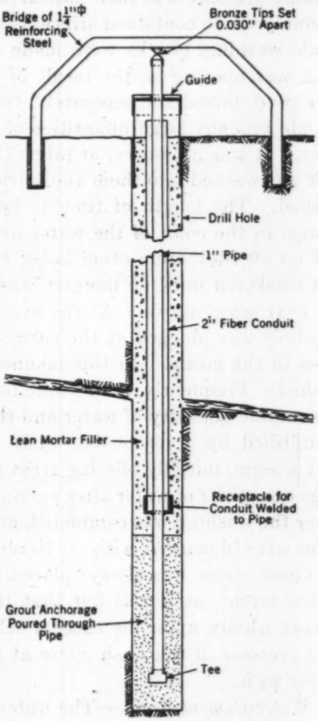

Bridge of 1¼"Φ Reinforcing Steel
Bronze Tips Set 0.030" Apart
Guide
Drill Hole
1" Pipe
2" Fiber Conduit
Lean Mortar Filler
Receptacle for Conduit Welded to Pipe
Grout Anchorage Poured Through Pipe
Tee

FIG. 17.—UPHEAVAL GAGE

A crew well trained in handling, and familiar with the limitations of the device in loose, crumbling, and otherwise unsound rock, is necessary to minimize trouble.

In washing a group of holes, the expander was always placed first on the lowest seam. The initial quantity of water that the seam would take at the washing pressure was then measured by passing it through a calibrated orifice assembly which contained orifices of different sizes to cover a wide range of flow. While washing, checks were made at regular intervals to learn whether the seam was opening as the result of the removal of material. Air and water were used, mixed and separately, to loosen the material, and the expansion of the air brought large quantities of muck to the surface from the connected holes. It was necessary, at intervals, to blow the nearby holes clear of muck that was washed into them and settled below the seam from which it was being washed. The length of time to be spent on each seam was judged by the change in the color of the water overflowing from connected holes. If there was no overflow from other holes, the rate of consumption of the wash water was measured until no increase was noted, when the expander was moved to the next seam above. As the overflow from any connected hole cleared up, this hole was plugged at the surface in order to force a passage to the other holes in the group. In this manner, each seam in each hole was thoroughly washed. Frequently, seams were found to be so tight that they would take no measurable quantity of water and the expander was moved when this fact was established by a reasonable trial. However, it was not uncommon to find that a seam, initially offering great resistance to flow, would open up and take large quantities of water after pressure had been maintained on it for some time. After the washing was completed, and just before the grouting was started, the holes were blown out with air to remove all muck.

Great stress was always placed upon the importance of thorough washing of the seams, as it was felt that the effectiveness of the grouting depended almost wholly upon the success achieved in removing the unsound material. The pressure of the wash water at the surface did not, in general, exceed 100 lb per sq in.

Water-Cement Ratio.—The water-cement ratio of the grout was determined in advance for each group, or for any individual holes not connected with others, by measuring the quantity of water which the holes would take at the washing pressure. For very tight holes the water-cement ratio was 3.0; for holes offering a fair degree of resistance, 1.5; and for holes that were very open, 0.66. What was termed the normal mix had a water-cement ratio of 1.0. This normal mix was used on holes that offered an average degree of resistance, and it possessed sufficient fluidity to be satisfactorily handled by the pumps. When the ratio was reduced below 0.66, the consistency of the grout was so great that trouble was experienced in handling it through long pipe lines. Extremely accurate control of the water was not considered necessary in view of the fact that much of the grout was pumped into water-filled seams.

CURTAIN GROUTING

After a group of holes had been washed and blown free of muck as described, short single expanders were placed in each hole. If connection with another group had developed during the washing, expanders were also placed in the holes of the other group. When a free connection between groups existed, every effort was made to grout them simultaneously with separate pumps. However, this was not always practicable and, in such cases, the lines were extended from one pump to both groups. Pumping would be started on one hole of a group at a rate of between 100 and 200 cu ft of cement per hr. When grout flowed from the other holes of the group, possibly many hours later, the overflow was stopped by means of valves on the expanders, and pumping was continued until overflow from the connected group resulted. The latter group was then closed off too, and the pressure on both groups was measured with the pump running. If the connected group showed appreciably less pressure than the group into which grout was being pumped, the flow of grout was diverted to the former, and from that time to refusal, the flow was changed back and forth between the two groups to maintain approximately the same pressure on both. When overflow failed to develop from a group with which interconnection was known to exist, the holes in this group were sampled at intervals by sounding with an open bottle to learn whether grout was leaking in. If the soundings indicated that it was coming in at a very low rate, the hole was kept open by blowing it out until a pump could be connected to it.

Pumping Rate.—On holes that offered no initial resistance to the inflow of grout, it was necessary to exercise judgment in determining the rate of pumping. In grouting seams of wide extent, pumping at a high rate may result in forcing large quantities of grout into remote regions where it is wasted in so far as the objective is concerned. A better effect might be obtained more economically by the judicious use of a much smaller quantity of thick grout, pumped slowly.

Upheaval Gages.—As a result of the great area covered by the seams underlying the foundation, the danger of raising the structure by the use of excessive pressures was a very real one, and upheaval gages were installed in the grouting gallery at 60-ft intervals for the length of the dam. The gage (see Fig. 17) consisted of a piece of 1-in. pipe with its lower end anchored by grout in the bottom of a vertical hole that had been drilled 10 ft deeper than the nearby grout holes. Above the anchorage, the pipe was encased in 2-in., asphalt-dipped, fiber conduit. After the pipe had been anchored by pouring grout through it, it was held in a vertical position by maintaining a strain on the top while the hole was filled with lean, coarse mortar. The strain was held until the mortar had set, thus assuring that the pipe would be restrained against excessive deflection as the result of carrying its own weight as a column. Across the top of the pipe, and set in the gallery floor, a bridge of 1.25-in. square reinforcing steel carried a bronze tip that was set approximately 0.030 in. from a similar bronze tip on the pipe. The gap between the tips was measured with a thickness gage at frequent intervals during grouting, and, if a progressive increase of as much as 0.010 in. occurred, it was construed as a warning that the safe pressure was being exceeded. When this occurred, the header was removed from the hole and the grout was allowed to flow out, the uplift gage being observed constantly to ascertain when settlement of the structure had ceased. When the original position of the structure had been assumed, or settlement had ceased, grouting was resumed at a reduced pressure.

Spacing and Depth of Holes.—In general, the holes for the curtain grouting were drilled 10 ft apart, but it was found desirable to reduce this spacing in some particularly bad areas to obtain a tight seal. Although this spacing might seem unnecessarily close to one who visualized extensive open seams through which grout could flow without interference, this condition did not exist in fact. In the writer's opinion, a plan view of any seam would have revealed that between 30% and 60% of the total area consisted of contact areas where no opening existed, and which, when penetrated by a drill hole, would not permit grout to flow into the seam. Thus, many of the holes drilled were not effective for grouting in some of the seams penetrated. This conclusion is based on observations made in a number of tunnels driven into seams, where it was noticeable that the openings frequently vanished entirely, only to reappear further along.

The locations of the principal seams were learned before the grout curtain holes were drilled, and therefore it was possible to determine, in advance, the depth to which the curtain should extend. To care for any local irregularities, holes were drilled in general 10 ft below the lowest seam known to underlie the location, or deeper if it was found that treatment was required at lower elevations.

Action of Grout.—In any grouting operation, great uncertainty prevails at all times as to just what is happening beneath the surface and as to whether the desired results are being obtained. The question of achieving economies by variations in methods arises constantly and must be judged with regard to the adverse effect upon the quality of the work that may result. If a uniformly open seam is pictured, the process is simple to analyze, but when one considers the infinite number of variations that may exist to defeat a preconceived notion, the uncertainty increases. Seams contain, in themselves, numerous channels, the easiest of which will be followed by the fluid grout, possibly for days. This fluid laps and splashes to form a hard coating over sand, clay, and unsound materials that may border the channel and, in addition, slowly deposits and builds over narrow passages leading from the channel. By the time that sufficient solid material has been deposited in the channel to offer enough resistance to flow to cause the grout to seek other passageways, these openings have been sealed over and will not permit the entrance of grout. The result of pumping a large quantity of grout into one hole may thus have been to fill one channel for a great distance without making much improvement on the condition of the general area. In some such manner, it frequently has happened that open passages have been left in a line of holes that were not spaced closely enough.

From observations made on numerous cores drilled from grouted seams and grouted holes, it appears that, in seams, the solid grout builds up slowly in layers as the cement is deposited. It is probable that this deposition begins when the velocity of the grout is reduced as it spreads out after leaving the drill hole. As continued deposition results in constriction of the passage, the cement is deposited farther and farther from the hole. Chemical affinity between the particles of cement is also an important factor in this building up of the solid material. Evidence of this fact may be seen in cores removed from redrilled grouted holes, where it frequently may be observed that the cement

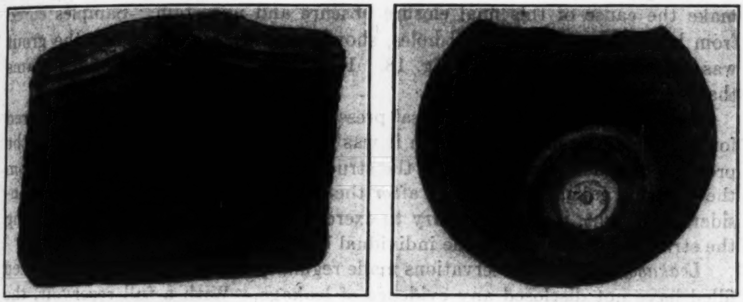

(a) Typical Seam (b) Typical Core

FIG. 18.—POLISHED SECTIONS SHOWING CHARACTERISTIC DEPOSITION OF GROUT

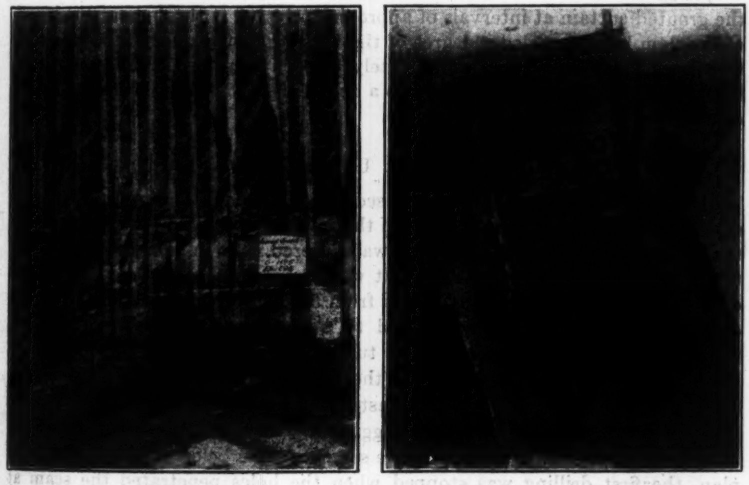

(a) Exposed by Excavation (b) In Large Core

FIG. 19.—TYPICAL GROUT SEAMS

has built up in concentric layers, gradually closing in and reducing the size of the passage. The possibility should not be overlooked that the drill hole may close before the seam plugs. The final closure probably results from the tendency of the cement particles in suspension to cohere to the deposited cement, gradually closing off the passageway. Many things may happen to make the cause of this final closure obscure and uncertain. Samples cored from both seams and grouted holes, showing the manner in which the grout was deposited, are shown in Fig. 18. Fig. 19 shows some grout-filled seams that were exposed after grouting.

Pressures.—In general, a refusal pressure of 150 lb per sq in. was required for the curtain grouting, although it was occasionally necessary to reduce the pressure in order to avoid lifting the structure. As this work was done from the grouting gallery in the dam, after the concrete had been poured to a considerable height, it was necessary to exercise great care to prevent damaging the structure by distorting the individual blocks.

Leakage.—Careful observations made regularly since the reservoir has been filled have not disclosed any evidence of leakage. With a full reservoir, the total flow measured in the stream bed at a point approximately 1 mile below the dam was 2 cu ft per sec. This quantity included the flow from a number of springs that existed before the dam was built. Test holes drilled to penetrate the grouted curtain at intervals of approximately 100 ft throughout the length of the dam have revealed a degree of tightness that was unexpected, the total flow from all holes being approximately 0.03 cu ft per sec. Nineteen drain holes, drilled in the spillway apron to a depth of 15 ft below the grouted zone, discharge a total of 0.55 cu ft per sec.

SEALING OF ROCK UNDER CORE WALL

From the east end of the gravity section of the dam, a reinforced concrete core wall was extended 584 ft through the rolled earth fill and overburden, and the grouting program along this core wall differed in no important respect from the methods used under the dam. It was necessary that the core drilling be done through overburden that varied from 50 to 100 ft in thickness. A large clay-filled seam (Fig. 20) was found to underlie this area at El. 965, and it was decided that a concrete-filled tunnel that followed the seam, directly beneath the core wall, would form the most economical and effective seal. Since it would have resulted in waste and unneeded duplication to have grouted this seam in addition to plugging it with concrete, the grouting was divided into two zones, one above the seam and one below it. Following this plan, the first drilling was stopped when the holes penetrated the seam at El. 965. Before grouting, thick dry mortar was poured into the holes to form a plug at the bottom that would prevent the passage of grout to the seam. The holes could not be stopped just short of penetration as the exact elevation at which the seam would be found was not known in advance. The seams in the upper zone were washed and grouted to refusal at a pressure of 75 lb per sq in.

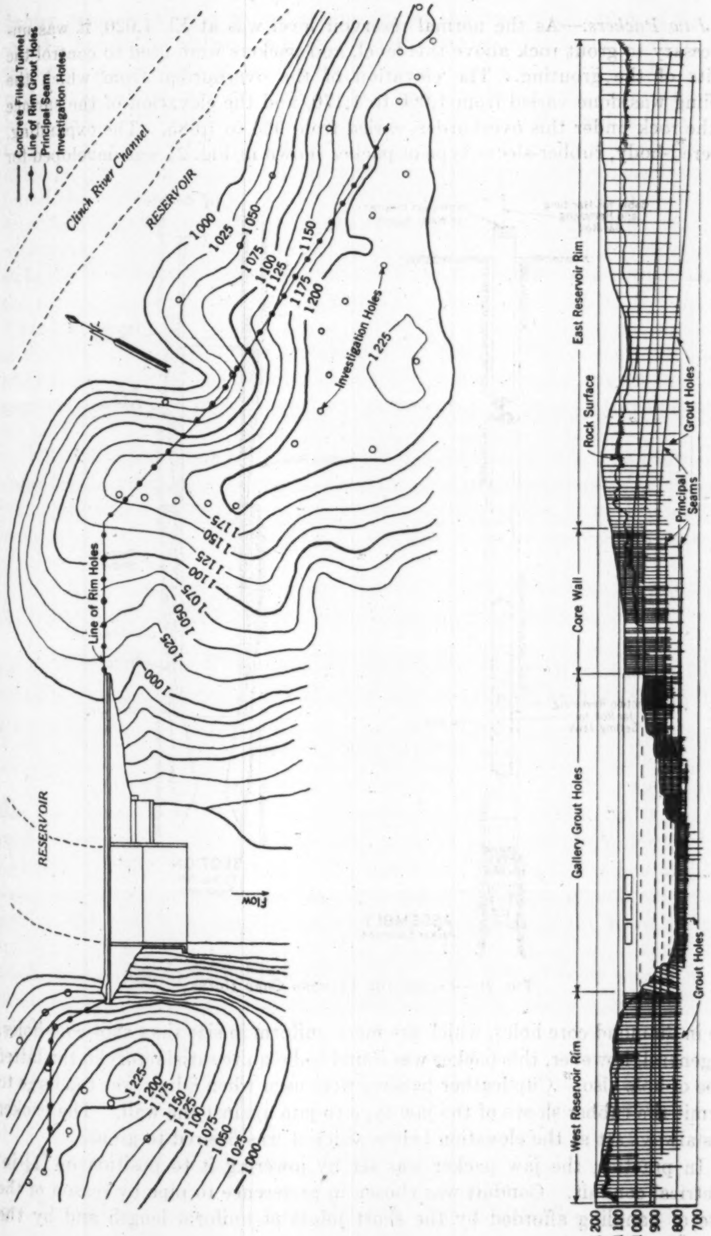

Concrete Filled Tunnel
Line of Rim Grout Holes
Principal Seam
Investigation Holes
Clinch River Channel
RESERVOIR
RESERVOIR
Line of Rim Holes
Investigation Holes
Flow
1 000
1 025
1 050
1 075
1 100
1 125
1 150
1 175
1 200
1 225
East Reservoir Rim
Rock Surface
Grout Holes
Principal Seams
Core Wall
Gallery Grout Holes
Grout Holes
West Reservoir Rim
Elevation in Feet
1 200
1 100
1 000
900
800
700

FIG. 20.—TOPOGRAPHY AND SECTION OF GROUTED AREAS

Jaw Packers.—As the normal reservoir level was at El. 1,020, it was unnecessary to grout rock above this level, and packers were used to control the limits of the grouting. The elevation of the overburden from which the drilling was done varied from 1,060 to 1,170, and the elevation of the surface of the rock under this overburden varied from 965 to 1,085. The expanding, tapered-body, rubber-sleeve type of packer shown in Fig. 21 was developed for use in diamond core holes, which are more uniform in size than shot core holes. In general, however, this packer was found to be quite satisfactory in the latter type of hole also. Cup leather packers were used when a hole was too large to permit the rubber sleeve of the jaw type to jam against the wall. The packer was always set at the elevation below which it was desired to grout.

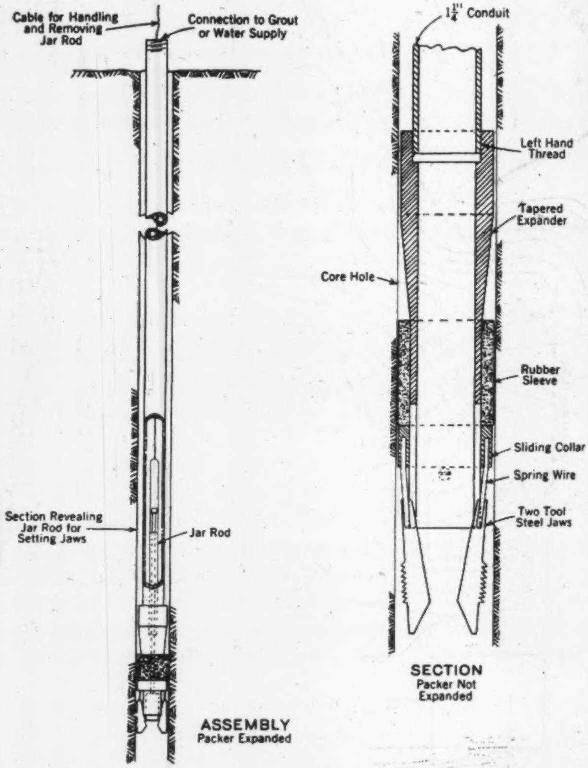

Cable for Handling and Removing Jar Rod
Connection to Grout or Water Supply
Section Revealing Jar Rod for Setting Jaws
Jar Rod
ASSEMBLY
Packer Expanded
1¼" Conduit
Left Hand Thread
Tapered Expander
Core Hole
Rubber Sleeve
Sliding Collar
Spring Wire
Two Tool Steel Jaws
SECTION
Packer Not Expanded

FIG. 21.—PACKER FOR DIAMOND CORE HOLES

In practice, the jaw packer was set by lowering it to position on 1¼-in. electrical conduit. Conduit was chosen in preference to pipe by reason of the ease of handling afforded by the short joints of uniform length and by the lighter weight. Having placed the packer at the desired elevation, a weight called a jar rod was lowered on a line until it rested upon the tapered jaws of the packer. The jar rod was made with a sliding body so that it could be driven quite forcefully against the jaws by raising and releasing the body with the handling line. By driving in this manner, the teeth on the packer jaws were forced to grip the wall of the hole and were held in that position by the weight of the jar rod. The conduit was then released slowly and its downward motion forced the rubber sleeve to expand upon the tapered body of the packer until it jammed against the wall of the hole and formed a seal. The jar rod was then withdrawn, as the friction of the rubber against the side of the hole was usually sufficient to support the column of pipe. When the friction was not sufficient, the weight was supported by a clamp that rested on the top of the casing. When grouting was completed, the packer was removed simply by pulling it out of the hole and was ready for use again immediately. To preclude the possibility of losing the conduit, should the packer become hung in any manner, the connection where it joined the packer was made with a left-hand thread so that the full length of conduit could be released by turning it clockwise.

When the upper zone had been grouted, a minimum of 24 hr was allowed to elapse before drilling was resumed to carry the hole to the final depth. Following the exploration and washing of the lower part of the hole, a packer was set immediately below the seam and the final grouting was completed at a pressure of 150 lb per sq in.

The labor cost of a grouting crew was as follows: 1 foreman at $1.00 per hr; 1 pump operator at $0.75 per hr; and 5 laborers at $0.45 per hr.

For grouting the dam foundations, 202,770 cu ft of cement grout at $1.521 per cu ft was required. The latter figure includes the cost of washing at $0.21 per cu ft of grout injected and the cost of pipe and fittings at $0.05 per cu ft of grout injected.

SHOT CORE DRILLING

For the curtain grouting of the dam and core wall, both 3-in. and 5.5-in. shot core holes were used. Genuine economy was obtained by using the smaller size only when it was necessary to drive casing through overburden. Although the initial investment in the smaller drilling tools was less and the shot consumption was less, the wear and consequent replacement was much greater and tended to counteract any economies that might have seemed apparent. Breakage of tools in the hole, with the resulting loss of time, was much more common with the smaller tools. The 5.5-in. holes served more effectively for exploration and seam washing and, in concrete or rock, cost little more than the 3-in. holes. The difference in cost of $1.026 per ft indicated in Table 1 is accounted for by the favorable effect of the large proportion of overburden drilling in the smaller size. Contributing to the foregoing unit cost was the labor cost of the typical core-drill crew, as follows: 1 driller at $1.00 per hr; 1 helper at $0.60 per hr; and 1 foreman at $1.25 per hr. The foreman supervised all drill crews.

Improvements in the rate of drilling, with a resulting reduction in costs, were secured by posting a daily record of each driller's footage and summarizing this at the end of the month to show the high men for the period. This was very effective in promoting a spirit of competition among drill operators and among shift foremen. Average drilling rates, costs, and other data are contained in Table 1.

TABLE 1.—DRILLING DATA

DRILL HOLE		EXTENT OF DRILLING, IN LINEAR FEET OF PENETRATION					UNIT COSTS, IN DOLLARS PER LINEAR FOOT	
Type	Diameter, in inches	Overburden	Rock	Total	Average depth	Average drilling rate, in feet per hour	Overburden	Rock
Wagon	2½	0	149,308	149,308	32	13.00	....	0.408
Shot core	3	4,532	15,363	19,895	245	1.28	....	2.634[b]
Shot core	5½	0	32,248	32,248	100	1.20	....	3.66
Diamond core[a]	2$\frac{1}{16}$	1,935	7,721	9,656	360	1.65	2.25	2.25
Diamond core[a]	2$\frac{5}{16}$	4,054	26,344	30,398	320	1.50	2.50	2.75
Well[a]	6¼	844	3,713	4,557	325	1.00	2.40	2.40

[a] Contracted. [b] Composite cost of overburden and rock.

RESERVOIR RIM INVESTIGATION

The geological survey made preliminary to the selection of the site revealed that the rim of the reservoir for distances of 9 miles from the east abutment and 5 miles from the west abutment would probably allow some leakage of water from the reservoir because of the geological characteristics of the formation. The cost of treating, or even of thoroughly investigating such a lengthy stretch of rim, would obviously have been prohibitive, and it was decided to confine the treatment to the narrow and definitely permeable parts adjacent to the abutments.

At the beginning of the rim investigation and treatment, the elevation of the water table underlying the areas in question was used as the criterion by which the permeability of the ridge was judged. That is, a high water table was interpreted as indicating that the rock beneath must necessarily be tight in order to support the water, and that, conversely, the rock above a low water table was sufficiently permeable to allow drainage of the ground water. Investigations must be sufficiently thorough to ascertain that a perched water table is not mistaken for the real water table. The first step, after determining the general areas to be investigated, was to drill 2$\frac{1}{16}$-in. diamond core holes approximately 200 ft apart on a line along the top of the ridge and to a depth reaching well below the bed of the river. The general layout of the investigation holes may be seen in Fig. 20. These holes were sounded daily for a sufficient length of time to determine the relation of the elevation of the water table to variations in the level of the water in the river. Where apparently open or pervious areas were found, as judged by the sensitivity of the table to changes in the elevation of the river water, other holes were drilled on each side of the ridge to establish a gradient and to ascertain that no tight barrier existed in that part of the ridge away from the reservoir. When the areas had been drilled as shown, daily soundings were made, extending over a period during which sufficient variation in river level occurred so that the characteristics of the holes were established definitely. By studying the hydrographs of the holes, it could be seen that from the beginning some followed closely every change in the elevation of the river water. Other holes never did fluctuate, whereas still others did not become active until the water rose to higher elevations, indicating the certainty that an open seam existed at the elevation at which the activity started. From the information obtained by sounding the holes, a contour map of the water table underlying the areas adjoining the abutments on each side of the river was made. This map, in conjunction with contour maps of the ledge rock and of the ground surface, was used to determine the location of the line of grout holes.

Diamond Core Holes.—During the drilling of the diamond core exploration holes, a detailed log of all findings, such as seams, core losses, loss of water, peculiarities of the rock, and any other pertinent information, was kept. However, the reliability of a large part of such information depends to a great extent upon the interest, vigilance, experience, and general ability of the drill operators, as well as upon the type and mechanical condition of the equipment. When using a screw feed machine, it is necessary (when the action of the machine indicates that a seam has been encountered) to release the chuck in order to measure the thickness of the seam accurately. This necessarily causes a delay in the contractor's operations, and too great a desire for progress will result in many seams being overlooked. Inattention and lack of interest on the part of the driller gives the same unsatisfactory results. The use of a double-tube core barrel, as compared with the single-tube barrel, reduces the core losses appreciably. A study of the records of the operations of two contractors, one using the former and one the latter for drilling in the same formation, revealed that the core loss was 5.9% with the double-tube barrel as compared with 24.9% with the single-tube. When drilling for grouting, the use of heavy compounds for lubricating the drill rods should be prohibited, as it is possible that this "rod dope" mixes with the cuttings to form a compound that seals small seams effectively against the entry of grout. Experience indicates that a diamond core drill, equipped with hydraulic feed, and using a double-tube core barrel, gives the most reliable results obtainable with small borings when information concerning subsurface openings is desired.

RIM GROUTING

Having determined the approximate location of the water table and of the rock surface in the areas in question and having obtained a general idea as to the frequency, location, and size of seams, it was next necessary to locate the lines upon which the grout holes were to be drilled. On the east side, in consideration of the length of the permeable portion, the distance of the region of low water table from the dam proper, and the character of seams and thickness of ridge, El. 1,020 (spillway crest) was set as the upper limit of the grouting, with holes spaced 50 ft apart. On the west side, because of the more cavernous nature of the rock, the close proximity to the abutment of the region of low water table, and the certain existence of large extensive seams, the upper limit of the grouting was raised to El. 1,050 through the most permeable portion, and the spacing of the holes was reduced to 20 ft. On both sides of the river, the holes were drilled to a depth that reached below river bed.

Location of Holes.—In locating the lines of grout holes then, it was necessary that the surface of the rock lie above the elevation of the upper limit of the grouting and desirable, in the interest of economy, to have the elevation of the ground surface as low as possible in order to minimize the amount of drilling. However, it was necessary to guard against apparent savings in depth secured by circumventing hills, as the resulting increase in the number of holes tended to offset any gains.

Since practically all of the holes were drilled from an elevation well above the upper limit of the grouting, it was desirable to avoid waste of the grout by confining it to the regions where it was needed. For this purpose, the previously described jaw packer shown in Fig. 21 was used.

On the west side, the line of holes was turned sharply upstream from the extended axis of the dam to avoid a large cave at El. 980. This cave opened on the bluff a short distance below the axis and extended back into the hill and across the axis. By grouting upstream from the cave, the cutoff was obtained more economically than it could have been secured by removing the loose material and filling the cave with concrete.

Water Tests.—After the packer was set at the desired elevation in a hole, a water test was made in an effort to gain some idea as to the quantity of grout that would be taken. The usual range of test pressures varied between 25 and 50 lb per sq in. This test was of value in that it was generally true that a tight hole would take little grout, whereas a very open hole could be expected to take large quantities. No rational basis for estimating the actual quantity of grout that would be required was determined.

Seams.—The elevation of the water table was considered highly important in determining the locations of the permeable portions of the rim, but, as more information concerning the location and size of the various seams was obtained from the drilling of the grout holes, the problem became more and more a matter of following and checking these seams carefully, regardless of the position of the water table. The position of the water table was obviously of little importance if seams (which, under greater head, would serve as drains) existed below it. Having determined the dip and strike of the rock accurately, it was possible to compute the approximate elevation at which a given seam should be found in any hole. By exercising unusual care when approaching the location at which the seam was expected to occur, the driller frequently could tell when it was penetrated. When no opening was found, the core loss often indicated the existence of unsound rock. Loss of drill water was also construed as a definite indication of an open seam.

Fig. 20 shows the seams that were traced from the ends of the dam through the rim on each side. The greater apparent angle of dip on the east side of the river resulted from a bend in the line of holes that placed the line nearer normal to the strike. The grouted curtain was ended on each side where the water table was found well above reservoir level. Although it was realized that it was quite possible that leakage would later occur through the rim, both within and beyond the grouted portions, it was felt that the expense of investigating and treating the entire part of the length that was questionable was not justified. It was decided, therefore, that the necessity for any additional treatment would be determined after the reservoir had been filled for some time and observations had revealed the seriousness of any leaks that might develop. Should leaks appear, drilling could be done with a full reservoir so that dye could be placed in the holes to determine when the path of the flow had been penetrated. In a number of cases, during the period of foundation treatment, fluorescein was used successfully for tracing subsurface flows. This dye may be traced in a very diluted solution if white porcelain containers are used for sampling the water.

The grouting of the reservoir rim differed from that of the dam foundations in several major respects. No attempt was made to wash unsound material from the seams, as it was felt that the small size of the $2\frac{5}{16}$-in. holes, spaced so widely, precluded the possibility of applying a sufficient volume of water to be effective. The objective was not a complete replacement of unsound material with grout, but a consolidation of this material by penetrating and filling the interstices with hard grout put in under sufficient pressure to compact the loose material in the seams. It was known that the openings in the rim were of such size and extent that, by maintaining a high rate of pumping, the grout could be forced to travel unneeded distances, increasing the consumption of material appreciably. Accordingly, the rate of pumping was usually limited so that not more than 80 to 100 cu ft per hr of solid material was handled by one pump.

Pressures.—In general, the refusal pressure for this work was limited to 25 lb per sq in. at the surface. As applied to holes of different depths, this was not an altogether consistent practice, as some variation in the pressure at the bottoms of the holes resulted. However, it was not thought necessary to use high pressures to obtain effective consolidation, although it was desirable to have some pressure on those seams which were near the top of the holes. The refusal pressure was set arbitrarily at 25 lb per sq in. at the surface in the belief that this pressure would afford the desired consolidation in the upper seams without causing undue waste in the lower regions. Since a general tightening of the rim was evidenced as the program neared completion, the pressure was sometimes raised in order to consolidate areas that were known to be unusually bad.

All studies following the preliminary investigations had tended to support the belief that large quantities of material would be required to consolidate the rim to the degree desired. These studies consisted of water tests of exploration holes and daily observation of the water levels in these holes to determine their sensitivity to fluctuations of the river level and to rainfall. Holes in which the elevation of the water always bore a close relation to the elevation of the river water, and that were little affected by rainfall, were considered as lying in open or permeable territory. Conversely, holes that, apparently, were not affected by river changes, and which were noticeably affected by rainfall, were considered as lying in tight rock.

USE OF ROCK FLOUR

With the thought that large quantities of material would be required and that a fine inert material might be used economically with cement to form satisfactory grout for rim tightening, a settling basin was built to collect rock flour from the sand classifier of the aggregate plant.

Preliminary Tests.—To determine the suitability of the rock flour cement mixture for grouting, preliminary field tests were made in an attempt to learn the characteristics of the resulting product. Samples that were taken from the mixer were observed to set and harden at a rate that indicated considerable retardation when compared with regular cement grout. Pumping tests proved that the characteristics of the mixture differed from those of cement grout to a degree that would make it desirable to revise the grouting program if its use was adopted, and established the necessity for laboratory tests supplemented by additional investigations in the field.

Laboratory Tests.—The laboratory tests indicated that the material was finer than cement, and a moisture-weight determination was made for use in proportioning mixes. The time-of-set data were perhaps the most interesting and pertinent as regards the determination of the suitability of the material for grouting, and it was definitely proved that the addition of rock flour had a retarding effect upon the setting time of the mixture.

Importance of Setting Time.—It may safely be assumed that, in grouting extensively seamed rock at a fixed rate of pumping, the area that will be covered is dependent to a large extent upon the setting time of the cementing material. By the use of a slow-setting material, the distance traveled by the fluid may be increased so that areas completely outside of the region that it is desired to treat will be grouted and the quantity of material necessary to effect consolidation increased appreciably. Obviously, even if the unit cost of the grout was reduced by the use of a cheap, inert material to replace part of the cement, the total cost of treating a given area might be increased if the quantity of material consumed was appreciably larger.

TABLE 2.—NORMAL CEMENT AND ROCK FLOUR

Test No.	Mix by Volume			Time of Setting, in Hours and Minutes			Test No.	Mix by Volume			Time of Setting, in Hours and Minutes		
	Cement	Rock flour	Water	Initial set	Final set	Soundness[c]		Cement	Rock flour	Water	Initial set	Final set	Soundness[c]
1	1	...	...[a]	2:10	5:00	OK	9	2	1	...[a]	2:50	6:35	OK
2	1	...	...[a,b]	0:40	2:00	OK	10	2	1	...[a,b]	1:45	3:25	OK
3	1	...	1.5	36:00	...	...	11	2	1	1.5	11:00	22:00	OK
4	1	...	1.5[b]	24:00	...	...	12	2	1	1.5[b]	4:05	10:30	OK
5	1	1	...[a]	3:15	6:50	OK	13	1	2	...[a]	4:30	6:45	OK
6	1	1	...[a,b]	2:45	5:00	OK	14	1	2	...[a,b]	2:15	4:20	OK
7	1	1	1.5	30:00	...	...	15	1	2	1.5	16:00	36:00	OK
8	1	1	1.5[b]	28:00	...	...	16	1	2	1.5[b]	10:00	18:00	OK

[a] Normal consistency (water 23% by weight of cement, or cement and rock flour). [b] $CaCl_2$, 3% by weight of cement, was added to the water. [c] Tests Nos. 3, 4, 7, and 8 did not attain final set in 6½ days. These mixes have an excess of water covering sample.

Addition of Calcium Chloride.—For these reasons, tests were made to determine the effect upon setting time secured by the addition of calcium chloride to the grout. The results of the complete series of tests are given in Table 2. It was found that the addition of this chemical had a tendency to correct the undesirable qualities of the rock flour mixture. It was learned that, by the addition of 3% of calcium chloride by weight of the cement, the set could be accelerated to a degree that would counteract, to a marked extent, the retardation resulting from the use of rock flour. The resulting product possessed characteristics of pumping and handling similar to those of the regular portland cement grout. In appearance, the product was little different from the regular grout, being dense and apparently impermeable, although not quite as hard. Specimens cored from grout-filled seams, after the rim grouting was under way, failed in compression at 2,000 lb per sq in. at an approximate age of 45 days.

Trial Run.—As a further step toward learning more of the properties of the rock flour grout, a test was conducted in which an effort was made to simulate the conditions surrounding the actual grouting of a seam. Four circular concrete slabs, ground to a flat uniform surface on one side, were matched in pairs, so that simultaneous tests could be run with the two types of grout. Temperature differences that might otherwise have affected the comparison were thus eliminated. Metal shims held the two slabs apart $\frac{1}{16}$ in. to form an artificial seam, and the grout entered this space through a 1-in. pipe passing through the center of the upper slab. A barrel in which the level of the grout was held constant by an overflow pipe, and into which the grout pump discharged, served to maintain a constant static head on the slabs. The pressure at the point where the grout entered the $\frac{1}{16}$-in. space was measured constantly during the test by means of a manometer. Throughout the duration of the test, and without interrupting the continuity of flow, the old grout was replaced by fresh grout at frequent intervals. A record of the temperatures of the fluids revealed that consistently higher temperatures prevailed in the regular cement grout. A number of tests were made but the full collection of data is not included in this paper. However, Table 3 summarizes the results obtained, and it is interesting to note that the addition of 3% of calcium chloride appreciably accelerated the setting of the rock flour cement mixture. Specimens to test for compressive strength were obtained from the mixer in the field, the mix being 1 part cement and 1 part rock flour. The water-cement ratio was 1.0. After 14 days the cylinders failed at an average of about 600 lb per sq in.

TABLE 3.—SUMMARY OF RESULTS OF ROCK FLOUR CEMENT GROUT TESTS

Grout mixture[a]	Hours required to grout space between slabs	Ratio of grouting time
2 cement: 2.5 water	19.25	1.00
1 cement: 1 rock flour: 2.5 water	64.30	3.34
1 cement: 1 rock flour: 2.5 water: 3% $CaCl_2$	30.30	1.57
2 cement: 2.5 water: 3% $CaCl_2$	9.50	0.49

[a] Proportion of water by volume; and proportion of calcium chloride ($CaCl_2$) refers to weight of cement.

The information obtained from these preliminary tests made it appear that considerable economy could be realized by using the rock flour cement grout containing calcium chloride, and a mixture containing equal proportions of cement and rock flour was adopted for use.

Handling Rock Flour.—It was learned after a trial run that the rock flour contained a quantity of fine clay that failed to disintegrate in the course of mixing in the regular manner. It also contained a small proportion of coarse material that made screening desirable. After some experimentation, it was found that the most satisfactory method of disintegrating the material was by means of a separate, mechanically-agitated mixer from which the rock flour passed in solution, through a screen, to the grout mixer. The mixers were very similar to those used for grout. The rock flour was hauled from the collecting basin as it was needed and did not have an opportunity to dry. The colloidal nature of the material apparently prevented drainage and, even in dry weather, it was necessary only to vibrate the apparently firm surface to reduce it to a jelly-like mass. For proportioning, the rock flour was measured by counting the number of shovels as the material was placed in the mixer, with an occasional check with a measuring box. Although this method seems crude, the nature of the material, its cohesiveness and resistance to breaking down into a granular form, made any other practical method very slow and difficult.

For the reservoir rim, 257,736 cu ft of rock flour cement grout at $0.535 was required.

Sand for Grouting.—During the preliminary grouting for sealing the first cofferdam, an attempt was made to obtain economy by mixing sand with the grout. The sand was placed in the mixer in an effort to handle it in the regular manner and, after a brief time, the pump and lines were plugged solidly. After this unsatisfactory trial, the use of this material was not attempted again until it became necessary to grout some holes in the rim that were known to penetrate unusually large openings. In this case, the holes took grout so freely that a vacuum existed at the surface and a funnel was installed on the header at the hole. Ordinary concrete sand was shoveled into this funnel where small water jets aided its flow. Knowing the rate at which the grout was being pumped, it was possible for the operator to proportion the mix by adding sand at a fixed rate. In this manner, the sand was handled quite satisfactorily, and a large quantity was placed in the two holes. However, the sudden and peculiar manner in which the holes refused raised some question as to the wisdom of accepting this material, and it was discontinued. Subsequent redrilling and regrouting of the holes that had refused to take more of the sand grout further confirmed the belief that it was unsuitable, as it was found that it was possible to pump a large quantity of the cement rock flour grout into the same seam that had been previously plugged with the sand mixture. Cores of the sand grout that were later removed from nearby holes indicated a strong tendency toward segregation, the material being lean and crumbly and of a generally poor quality. No doubt segregation could be reduced by the use of find sand. In general, the writer believes it unwise to use sand for the primary grouting of seams in a foundation that will be subjected to more than a moderate head.

CONCLUSION

The treatment of every foundation produces new and unforeseen problems to which judgment and experience offer the best solution. By reason of its nature, the work does not lend itself to control by rigid specifications and, once the broad plan of approach has been determined, the responsibility for the changes that will be necessary in order to cope with the various emergencies that will arise is best placed upon the field forces. It is seldom that the quality of the work can be permitted to suffer from the delays incidental to consultations and special studies, and the changes are usually in the nature of emergencies. For this reason, it is well to choose inspectors possessing initiative and judgment for directing the work.

The uncertain character of the work makes it impractical to estimate the cost of foundation treatment in advance with any assurance that the values obtained will be better than a guess. It is true that, although subject to wide variation, unit costs may be approximated, but it is difficult to approximate the total quantity of materials that will be required. Having no basis of comparison, therefore, it is not easy to judge whether the work is being performed economically. However, by keeping the object of the work in mind constantly and by so regulating the pumping rates and fixing the refusal pressures that the least material commensurate with the maintenance of the desired standard of quality is used, appreciable economies may be effected. In practice, this means that a study must be made to determine the characteristics of every hole or group that is to be grouted and that pressures, pumping rates, and water-cement ratios must be varied to suit the individual peculiarities of each.

Modern methods of foundation treatment leave much to be desired from the standpoint of both effectiveness and cost, and the obscure nature of the work often causes it to receive less attention than its expense warrants. It is a subject in the interest of which much study and experimentation could profitably be undertaken, as it is probable that, as available sites are utilized, it will be found frequently that the foundations present greater problems than the structures. The discovery of suitable cheap materials, inert or otherwise, and in a form to be mixed with cement or to be used alone, would be a great step forward in the field of foundation treatment. The extended use of chemicals for grouting or for varying the properties of conventional grout mixtures also has interesting possibilities for obtaining economies through better control of the limits of the work. The manufacturers of drilling equipment have made noteworthy improvements in their products and in the development of new aids to drilling in recent years, and this progress is reflected in the lower drilling costs that prevail today. In the future, the major economies that will be effected in the field of foundation treatment will probably result from the development of cheaper materials for grouting.

ACKNOWLEDGMENTS

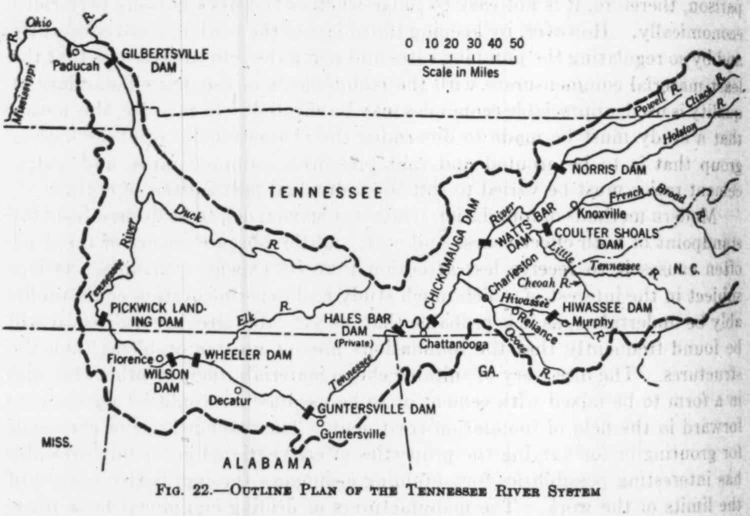
The writer wishes to acknowledge gratefully the valuable help received from both the construction and engineering forces of the TVA in performing this work with the exactitude and attention to detail that was necessary. Credit is especially due to G. A. Carlson and J. C. Evans, Jun. Am. Soc. C. E., for their assistance in directing the work, as well as for their aid in preparing this paper.

FOUNDATION EXPLORATION AND GEOLOGIC STUDIES AT GUNTERSVILLE DAM

By Robert M. Ross,[2] Esq.

Synopsis

The construction of a dam in the vicinity of Guntersville, Ala. (see Fig. 22), has been under consideration since 1914. Geologic investigations of several proposed sites were made in that year by the U. S. Army Engineers, and the work was continued in 1933. The first investigations by the Tennessee Valley Authority (TVA) were begun in the fall of 1934. A detailed geologic map of the region was made, stratigraphic studies of the various formations involved were conducted, numerous geologic sections were measured and described, and considerable structural data were accumulated. Eight dam sites were examined. Each was subjected to an intensive study, which was greatly facilitated by the diamond-drill cores previously obtained by the Army Engineers. This paper is a brief summary of the geological studies upon which the final design was based.



Ohio
R.
R.
Mississippi
Paducah
GILBERTSVILLE DAM
0 10 20 30 40 50
Scale in Miles
Powell R.
Clinch R.
R.
Holston
NORRIS DAM
TENNESSEE
French Broad
R.
Knoxville
Duck
River
WATTS BAR DAM
COULTER SHOALS DAM
CHICKAMAUGA DAM
River
R.
Little
Tennessee
R.
N. C.
Tennessee
Charleston
Cheoah R.
Hiwassee
HIWASSEE DAM
PICKWICK LANDING DAM
Elk
R.
HALES BAR DAM (Private)
Reliance
Murphy
R.
Chattanooga
Ocoee
Florence
WHEELER DAM
Tennessee
R.
GA.
WILSON DAM
Decatur
GUNTERSVILLE DAM
Guntersville
MISS.
ALABAMA

Fig. 22.—Outline Plan of the Tennessee River System

Regional Geology

The rocks in the Guntersville area are all sedimentary, and all belong to the Paleozoic system. During past geologic time they were subjected to powerful compressive stresses from the southeast which folded them into a great arch, known as the Sequatchie anticline. The arching of the strata caused them to crack and to become highly susceptible to the forces of weathering so that the anticline became eroded until today it is expressed topographically as a deep valley. The Sequatchie Valley is 4 or 5 miles wide at Guntersville; it continues south about 20 miles and extends far into the State of Tennessee, to the north.

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Because of the folding and the erosion which followed it, the various formations present outcrop in long, parallel belts. The middle part of the valley is occupied by the relatively soft Chickamauga limestone, which forms low, flat areas. On either side of the Chickamauga limestone, forming a pair of straight, much dissected, subordinate ridges, are the Red Mountain formation, which is mainly limestone and calcareous shale, the Chattanooga black shale, and the Fort Payne chert (see Fig. 23). The soluble Bangor limestone occupies the valleys between the ridges and the bordering escarpments and composes the lower part of the escarpments. The resistant Pottsville formation caps the high plateau areas which lie on either side of the valley.

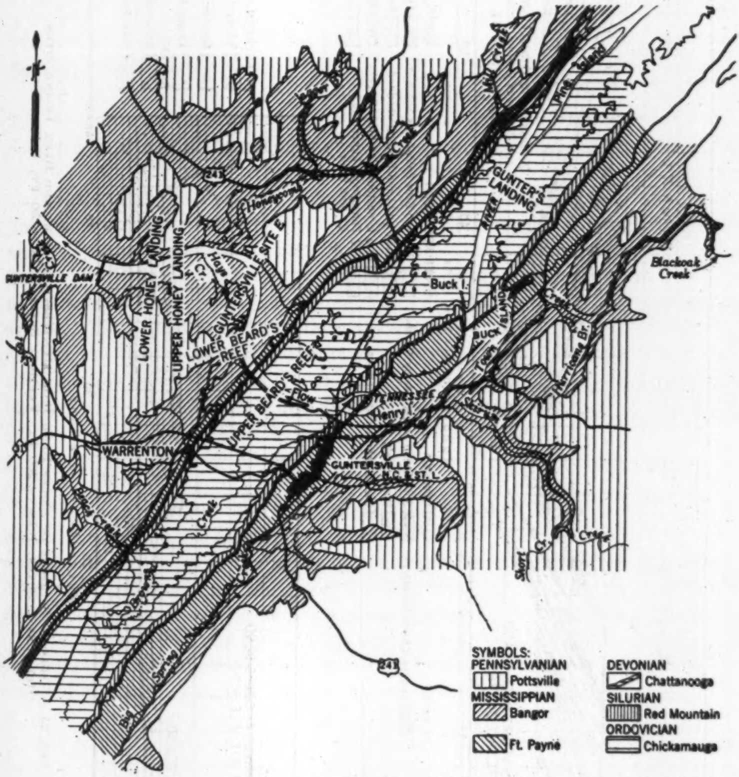
Mill Creek
Pine Island
Creek
241
Honeycomb
GUNTER'S LANDING
RIVER
GUNTERSVILLE DAM
LOWER HONEY LANDING
UPPER HONEY LANDING
Hogs Cr.
GUNTERSVILLE SITE E
Blackoak Creek
Buck I.
Creek
BUCK ISLAND
Town Creek
Hurricane Br.
LOWER BEARD'S REEF
UPPER BEARD'S REEF
Flow
TENNESSEE
Henry I.
WARRENTON
GUNTERSVILLE
N.C. & ST. L.
Creek
Short Cr.
Creek
Big Spring Creek
241
SYMBOLS:
PENNSYLVANIAN
Pottsville
MISSISSIPPIAN
Bangor
Ft. Payne
DEVONIAN
Chattanooga
SILURIAN
Red Mountain
ORDOVICIAN
Chickamauga

FIG. 23.—GEOLOGIC MAP OF GUNTERSVILLE AREA, SHOWING ALTERNATE DAM SITES

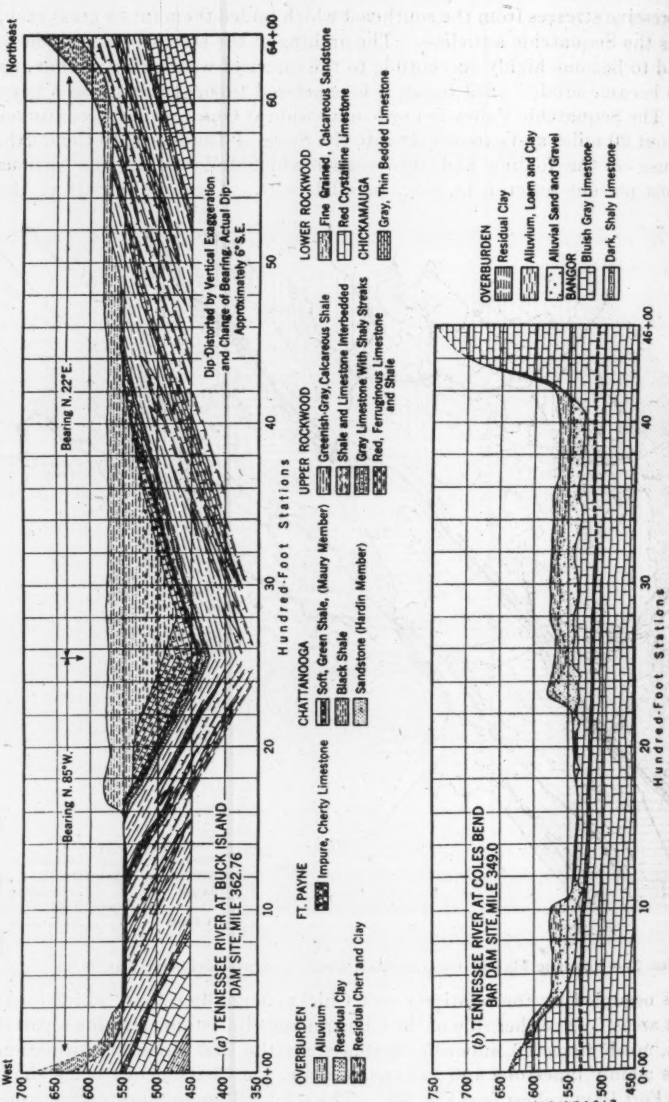
West
Northeast
Bearing N. 85° W.
Bearing N. 22° E.
Dip Distorted by Vertical Exaggeration and Change of Bearing. Actual Dip Approximately 6° S.E.
(a) TENNESSEE RIVER AT BUCK ISLAND DAM SITE, MILE 362.76
Elevation, in Feet
700
650
600
550
500
450
400
350
0+00
10
20
30
40
50
60
64+00
Hundred-Foot Stations
OVERBURDEN
Alluvium
Residual Clay
Residual Chert and Clay
FT. PAYNE
Impure, Cherty Limestone
CHATTANOOGA
Soft, Green Shale, (Maury Member)
Black Shale
Sandstone (Hardin Member)
UPPER ROCKWOOD
Greenish-Gray, Calcareous Shale
Shale and Limestone Interbedded
Gray Limestone With Shaly Streaks
Red, Ferruginous Limestone and Shale
LOWER ROCKWOOD
Fine Grained, Calcareous Sandstone
Red Crystalline Limestone
CHICKAMAUGA
Gray, Thin Bedded Limestone
(b) TENNESSEE RIVER AT COLES BEND BAR DAM SITE, MILE 349.0
Elevation, in Feet
750
700
650
600
550
500
450
0+00
10
20
30
40
46+00
Hundred-Foot Stations
OVERBURDEN
Residual Clay
Alluvium, Loam and Clay
Alluvial Sand and Gravel
BANGOR
Bluish Gray Limestone
Dark, Shaly Limestone

FIG. 24.—GEOLOGIC SECTIONS OF COLES BEND BAR (GUNTERSVILLE DAM) AND BUCK ISLAND SITES (HORIZONTAL SCALE, 1 IN. = 200 FT; VERTICAL SCALE, 1 IN. = 50 FT; V.E. = 4)

SELECTION OF THE SITE

Six of the eight original sites were eliminated from consideration after the preliminary studies. They were all in soluble limestone formations, and at each site the foundation rock had suffered extensive solution and contained numerous cavities. This condition made the sites very unsuitable for the erection of a dam.

The two remaining sites were the Buck Island site, about 4 miles upstream from the town of Guntersville, and the Coles Bend Bar site, 9 miles downstream. A thorough diamond-core drilling program was begun at both sites in July, 1935. The work at Buck Island was completed in September and that at Coles Bend Bar in the following May (see Fig. 24).

The Buck Island site was originally located in the Chickamauga limestone. This was too cavernous to be acceptable so the site was transferred 0.7 mile downstream, where the more resistant, ridge-forming rocks crossed the river in narrow belts. It was hoped that the Red Mountain formation, the Chattanooga black shale, and the Fort Payne chert might provide a suitable foundation. The first two came up to expectations, but the Fort Payne, although it forms a relatively sound, resistant foundation for both Wilson and Wheeler dams, was deeply weathered at Buck Island and utterly unsuitable. At some points it had deteriorated to a mass of loose chert and clay to depths of 140 ft below the surface. The satisfactory formations were so narrow that it was impracticable for them to support the entire dam. In contrast, the rock at Coles Bend Bar, although limestone, was found to be unusually sound. These conditions, together with other factors, such as the added cost of dredging to the Buck Island site, resulted in selecting the Coles Bend Bar site.

THE GUNTERSVILLE DAM SITE

The axis of Guntersville Dam crosses the Tennessee River at right angles, is 3,985 ft long, and has a bearing of North 18° East. The central part is of the concrete gravity type and consists of a navigation lock, spillway, and power house. On either side of the river, flanking the masonry section, are earth embankments which extend across the flood plains to the abutments.

Topography.—The north flood plain is about 600 ft wide and has an elevation of from 29 to 33 ft above low water. The south flood plain is about 1,800 ft wide and has an elevation of 21 to 37 ft above low water. Both abutments are at the bases of steep hills which rise 500 ft or more above the river.

Geology.—Throughout most of the flood plain the alluvium is roughly divisible into an upper part consisting of loam and clay and a lower part consisting of sand, or sand and gravel. The loam and clay, according to results obtained from earth auger borings, averages slightly more than 10 ft in thickness. It is relatively impervious. The underlying sand and gravel is usually between 20 and 25 ft thick and is often very permeable, especially just above bedrock. In order to intercept the flow through these permeable areas and prevent possible leakage, it was necessary to drive steel sheet piling to bedrock along the entire lengths of both earth embankments. At the abutments, the alluvial material gives place to a heavy, brownish-red, residual clay which is extremely stiff and impervious.

The site is located in the middle portion of the Bangor limestone. The formation rises about 350 ft above river level and probably extends downward as much as 300 ft. The foundation rock is uniformly hard, massively bedded, rather dark bluish-gray, crystalline limestone, except for a relatively thin stratum of shale and shaly limestone. The shaly stratum has an average thickness of about 3 ft. It is of wide horizontal extent, there being only two small areas in the vicinity of the site where it appears to be absent. One of these is at the left bank of the river and the other extends beneath the flood plain from the left abutment.

The bed is divisible, characteristically, into three members. The upper and lower are black or dark gray, thin-bedded, considerably weathered, carbonaceous shale. The middle member is dark gray, compact, shaly limestone. It is likely that, originally, the entire stratum was shaly limestone, but the upper and lower portions, being next to the bedding planes and more accessible to percolating water than the middle, had part of their calcareous substance leached out and the residuum compacted and converted into shale. Chemical analyses reveal the bed to be much lower in carbonates, and consequently much less soluble, than the limestone above and below it.

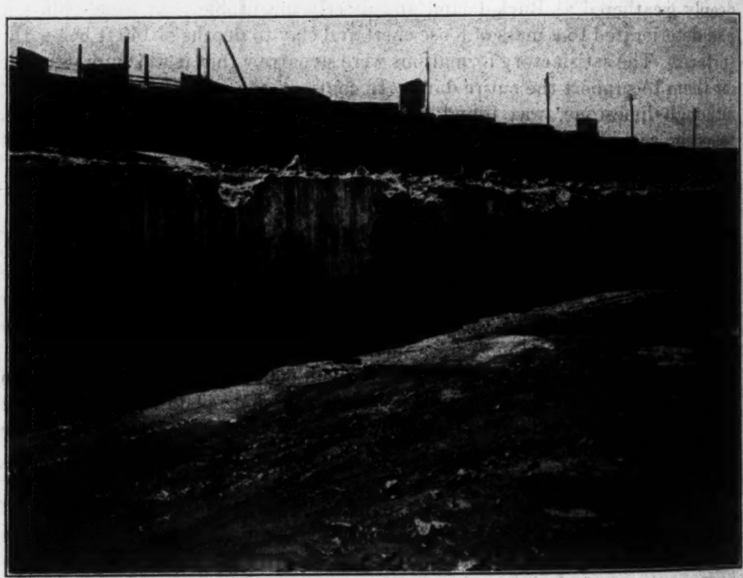

FIG. 25.—SHALY STRATUM EXPOSED IN LOCK SECTION, GUNTERSVILLE DAM

The shaly stratum has had a profound influence on the foundation rock and is probably responsible for its unusually good condition (see Fig. 25). Since it is relatively impervious, it has served to shield the strata below it from percolating waters and hence to prevent solution. Because of its presence, the deep seams and cavities which ordinarily occur in the Bangor limestone, beneath the river, are very largely absent. Along the axis the stratum is overlain by from 2 to 30 ft of limestone. In some areas of this rock, solution is extensive, whereas below the shaly bed the rock is remarkably sound.

The effectiveness of the bed in protecting the rock below it is demonstrated graphically by the logs of the diamond-drill holes at the dam site. Altogether, 186 holes (110 of which penetrated the horizon of the shale) were drilled under the direction of the TVA; 264 seams were present, and of this number 245 were above the level of the shaly bed. Of the 19 that occurred below it, 15 were in areas where the shaly bed was indistinguishable and only 4 were below a recognizable shale.

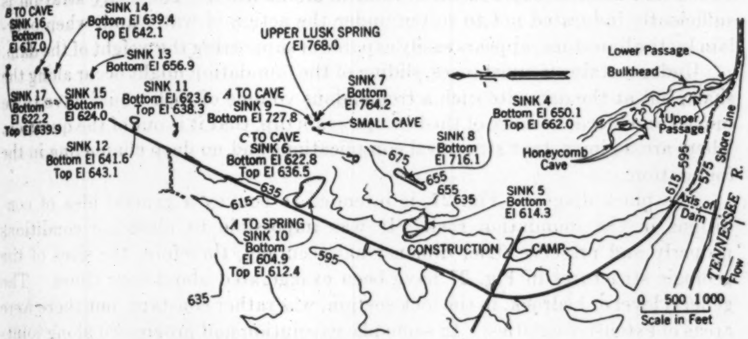
B TO CAVE
SINK 16
Bottom
El 617.0
SINK 14
Bottom El 639.4
Top El 642.1
SINK 13
Bottom El 656.9
SINK 17
Bottom
El 622.2
Top El 639.9
SINK 15
Bottom
El 624.0
SINK 11
Bottom El 623.6
Top El 638.3
SINK 12
Bottom El 641.6
Top El 643.1
UPPER LUSK SPRING
El 768.0
A TO CAVE
SINK 9
Bottom El 727.8
SINK 7
Bottom
El 764.2
SMALL CAVE
C TO CAVE
SINK 6
Bottom El 622.8
Top El 636.5
A TO SPRING
SINK 10
Bottom
El 604.9
Top El 612.4
SINK 8
Bottom
El 716.1
SINK 4
Bottom El 650.1
Top El 662.0
Lower Passage
Bulkhead
Upper Passage
Honeycomb Cave
SINK 5
Bottom
El 614.3
CONSTRUCTION
CAMP
Axis of Dam
Shore Line
TENNESSEE R.
Flow
675
655
655
635
635
615
615
595
595
575
635
0 500 1000
Scale in Feet

FIG. 26.—HONEYCOMB CAVE AND RELATED SINKHOLES

The bedding is practically horizontal. There is a dip to the south along the axis of about one-fourth degree. This is not the true dip since the axis is not parallel to the strike. The strike is approximately North 35° East and the regional dip is something less than 1° Southeast, or upstream, although there are slight irregular undulations in the rock which cause local variations of 1° or 2°.

Occasional slickensides and small calcite veins, indicative of pressure from the southeast in times past, may be seen. These are too slight to have any practical significance. There are three distinguishable joint sets. Two are formed of shear joints, produced by compressional stresses. One of these has an approximate trend of from North 10° West to North 25° West; the other trends between North 70° West and North 85° West. The former is the dominant set. The caves and the solution channels in the vicinity usually follow it, having occasional cross branches developed along the second set. The third set is composed of strike joints which vary considerably but nearly all lie between North 10° East and North 30° East.

Condition of the Foundation.—The effects of solution constituted the only serious foundation problem at the dam. The limestone contained an average of more than 90% carbonates and sometimes more than 98%. As a result, cavities and enlarged joints were numerous in some areas, although the shaly stratum, which may be considered the lower limit of appreciable solution, confined them largely to the surface beds (see Fig. 26).

With the exception of the shale, the foundation rock is hard and resistant to stress. Compression tests on a number of samples revealed an average crushing strength greater than 23,000 lb per sq in. Samples of the weathered shale were far weaker, averaging only 155 lb per sq in.; but it must be remembered that the shale tends to soften and crumble on exposure to the air, and the samples tested had been so exposed for several hours at least. The shaly stratum is sufficiently indurated not to soften under the action of water and, when overlain by the limestone, appears easily capable of supporting the weight of the dam.

Under certain circumstances, sliding of the foundation might occur along the shale, but at the dam site such a tremendous volume of rock would have to be moved and lifted, because of the low upstream dip, that it is out of the question. There are no important structural complications and no deep channeling in the foundation.

The block diagram, Fig. 27, is intended to convey a general idea of conditions in the foundation rock. It was impossible to illustrate conditions properly and preserve strict dimensional accuracy; therefore, the sizes of the geologic structures in Fig. 27 have been exaggerated about four times. The general level of bedrock, in the lock section, was rather constant, but there were areas of extensive cavities. In some places solution had progressed along joints and bedding planes until a system of crisscrossing, vertical and horizontal openings, was produced. Most of the vertical seams were developed along joints. Such seams ran in more or less straight lines, cutting across each other at high angles, so that the rock was sometimes divided into numerous rhombic

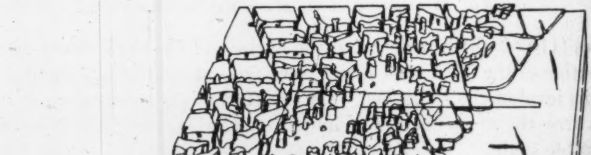

Fig. 27.—Block Diagram of the Original Rock Foundation at Guntersville Dam Site ($V E = 4$)

blocks. These were often less than 4 ft in diameter, where solution was far advanced. The horizontal seams developed along the bedding planes. They intersected the vertical seams more or less at right angles and gave rise to a zigzag, step-like system of cavities which often exceeded 5 or 6 in. in thickness and sometimes attained thicknesses of 3 or 4 ft. They were most abundant near the surface and diminished rapidly with depth. Most of them occurred in the upper 8 ft, and there was a definite stratum of highly dissolved rock between 2 and 6 ft below the surface.

Because of the advanced solution in the rock overlying the shaly stratum, the lock foundation was excavated to the surface of the underlying limestone, which was very sound and unweathered. Farther south, in the spillway section, the rock became very smooth and free from seams. There were relatively few open joints, even at the surface, and those that did occur were not very deep.

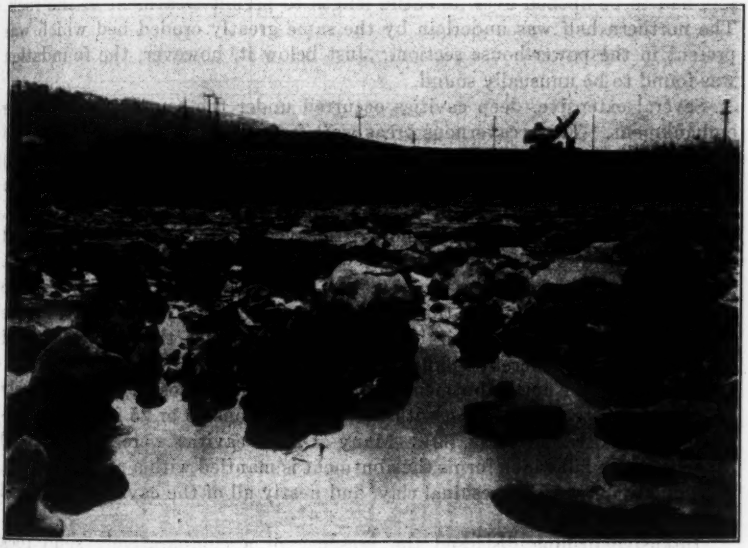

Fig. 28.—Eroded Stratum in Power-House Section, Guntersville Dam

The dip is so low that the upper part of the foundation throughout most of the section was composed of a single stratum. In the southeastern part a thin, overlying bed about 2 ft thick appeared. A third occurred above the other two, still farther to the southeast, where the strata were lowest. It was from 4 to 6 ft thick and had suffered such erosion that it was reduced almost everywhere to discontinuous isolated blocks and boulders, surrounded by alluvium.

The same three beds continued on into the power-house section. The highest, consisting of a multitude of eroded remnants, covered more than half the section, whereas the lower ones outcropped in the northwestern part (see Fig. 28). The two upper beds were removed, and the lowest provided an unusually good foundation, notwithstanding the fact that its surface was 18 or 20 ft above the shaly stratum.

Thorough core drilling was done throughout the entire masonry section, both for exploration and for grouting and drainage. All the holes were examined carefully and logged. A number of 36-in. holes were drilled, thus permitting a careful visual inspection of the foundation rock to depths of 30 ft or more. After excavation was completed, a detailed map of all joints in the rock was made so that every path for potential leakage would be known in the event that further foundation treatment should ever become necessary.

Diamond core drilling indicated that the rock beneath the north embankment was relatively free from seams, and this conclusion was confirmed subsequently by the holes bored for grouting. The rock surface was smooth except for a few irregularities near the abutment where several residual boulders were present.

The south embankment foundation was much less satisfactory. Ultimately, pits had to be opened over its entire length to permit treatment of the rock. The northern half was underlain by the same greatly eroded bed which was present in the power-house section. Just below it, however, the foundation was found to be unusually sound.

Several extensive, deep cavities occurred under the southern part of the embankment. These cavernous areas were from 27 to 138 ft long, along the axis, and sometimes went as deep as the shaly horizon, more than 25 ft beneath the top of the bedrock. They appeared to continue upstream and downstream for considerable distances and were always partly filled with sand and gravel. The cavity in Fig. 29 was much more extensive than indicated by the shaded area, which shows only the part that was actually excavated and explored. This cavity was cut off by a concrete wall and iron piles. The cavities were ordinarily between 3 and 6 ft thick, but the maximum was nearly 12 ft. Since grouting was considered undesirable in openings of this size, they were excavated and blocked with concrete bulkheads.

The rock in the north abutment was very much decomposed. An extremely widespread system of cavities, with bottom elevations 20 or 25 ft below flood-plain level, existed (see Fig. 30). Many of these cavities were more than 20 ft thick. The hill which forms the abutment is mantled with a reddish-brown, stiff, highly impervious, residual clay, and nearly all of the cavities were filled with this material.

Intensive drilling disclosed the presence of a comparatively solid rock projection in the abutment about 50 ft upstream from the axis. In order to intercept the cavities, a trench was excavated to sound rock, and a concrete cutoff wall built from the axis to the projection. The abutment was later grouted thoroughly.

The south abutment is very broad, steep, and solid. The rock is unusually sound, although its surface is cut by numerous shallow, vertical fissures, and several thin horizontal seams are developed along bedding planes. Ordinarily, these were filled with heavy, residual clay so that leakage appeared unlikely. Nevertheless, the rock was explored carefully with drill holes and grouted.

Honeycomb Cave.—Of all the caves in the vicinity of the dam, only one was so situated that it might cause any trouble. This cave, known as Honeycomb Cave (see Fig. 26), is in the plateau remnant which forms the right abutment. The mouth of the cave is about 2,000 ft upstream from the dam

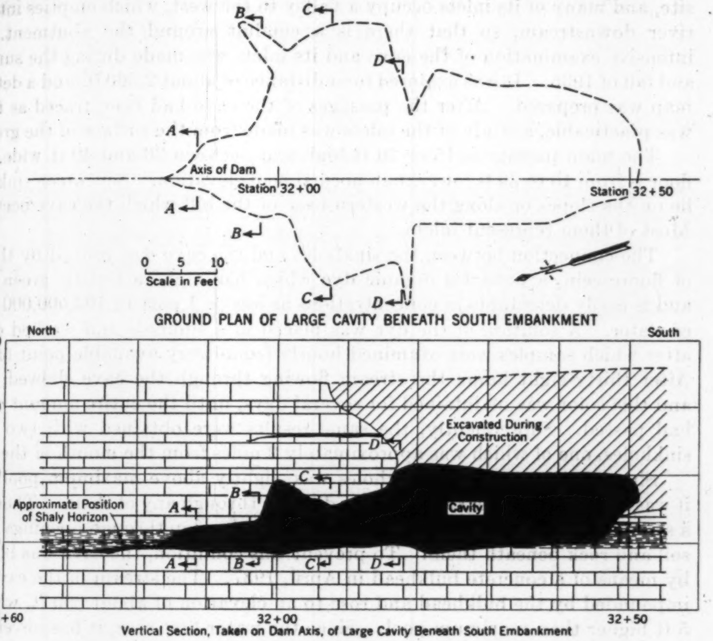
A
B
C
D
Axis of Dam
Station 32+00
Station 32+50
0 5 10
Scale in Feet
GROUND PLAN OF LARGE CAVITY BENEATH SOUTH EMBANKMENT
North
South
540
530
520
510
500
Elevation, in Feet
Excavated During Construction
Approximate Position of Shaly Horizon
Cavity
31+60
32+00
32+50
Vertical Section, Taken on Dam Axis, of Large Cavity Beneath South Embankment

FIG. 29.—LARGE CAVITY IN FOUNDATION ROCK BENEATH THE SOUTH EMBANKMENT

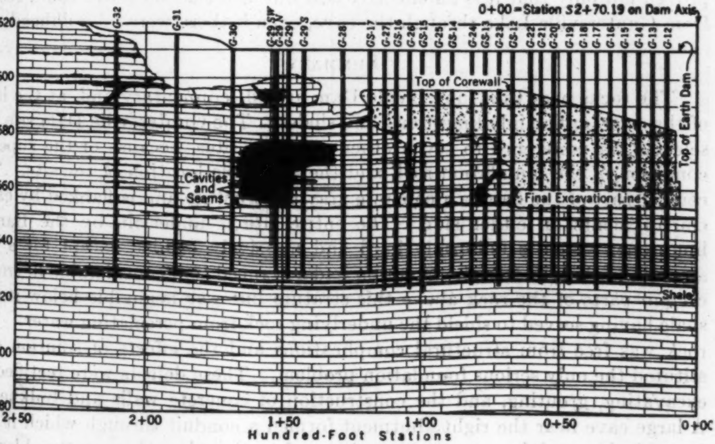
0+00 = Station 52+70.19 on Dam Axis
G-32
G-31
G-30
G-29
G-28
G-27
G-26
G-25
G-24
G-23
G-22
G-21
G-20
G-19
G-18
G-17
G-16
G-15
G-14
G-13
G-12
Top of Corewall
Top of Earth Dam
Cavities and Seams
Final Excavation Line
Shale
620
600
580
560
540
520
500
480
Elevation, in Feet
2+50
2+00
1+50
1+00
0+50
0+00
Hundred-Foot Stations

FIG. 30.—CAVITIES AND LOCATION OF DRILL HOLES; VIEW FACING UPSTREAM ALONG AXIS OF NORTH ABUTMENT, GUNTERSVILLE DAM

site, and many of its inlets occupy a valley to the west, which empties into the river downstream, so that there is a conduit around the abutment. An intensive examination of the cave and its inlets was made during the summer and fall of 1935. It was explored for a distance of about 2,500 ft, and a detailed map was prepared. After the passages of the cave had been traced as far as was practicable, a study of the inlets was made from the surface of the ground.

The main passage is 15 or 20 ft high and between 20 and 40 ft wide. Its floor is from 16 to 39 ft lower than normal pool elevation. Seventeen sinkholes lie on the slopes or along the western base of the hill which the cave occupies. Most of them represent inlets.

The connection between the sinkholes and the cave was proved by the use of fluorescein, a powerful organic dye which has a characteristic green color and is easily detectable in concentrations as low as 1 part to 100,000,000 parts of water. A solution of the dye was placed in a sinkhole and washed down, after which samples were examined hourly from every available point of exit. After thirty-eight hours the stream flowing through the cave showed color, and this color was maintained for several days, until the entire amount of dye had passed. Subsequently, the same results were obtained with two other sinkholes, one of which was approximately 2 miles from the mouth of the cave.

Since the lowest of the sinkholes was slightly above maximum pool level, it was impossible for leakage to occur directly through any of them. There was a strong possibility, however, that leaks might develop through openings in the soil and rock beneath them. To prevent this condition, the cave was blocked by means of a concrete bulkhead in April, 1937. The stream in the cave was impounded by the bulkhead and rose to an elevation of about 600 ft, which is 5 ft higher than maximum pool. Since the water first rose, it has never been lower than about 590 ft and has stood above 595 ft for months at a time without any leakage into the valley below the dam. Consequently, the bulkhead has proved not only that it is an effective seal but also that no water could escape from Guntersville Lake through the cave, even if there were no bulkhead.

SUMMARY

The rocks near the Guntersville Dam are all sedimentary and, at the level of the river, are mainly limestones and shales. The Guntersville Dam site was selected, in preference to seven other proposed sites, because of the superior condition of the foundation rock and the favorable location of the site. The central part of the dam is a concrete gravity structure and is flanked by earth embankments on either side. The entire site is underlain by the Bangor limestone, which is uniformly hard and massively bedded except for a 3-ft stratum of shaly material. Solution had advanced to a considerable degree in certain parts of the rock above this stratum but was negligible below it, the shale having served to shield the underlying rock from percolating water. The rock was free from structural complications and the effects of solution constituted the only serious foundation problem. These defects were rectified by excavating, grouting, and the construction of concrete walls and bulkheads. A large cave near the right abutment formed a conduit through which it was feared leakage might occur. After thorough exploration, the cave was blocked effectively with a concrete bulkhead.

FOUNDATION CONDITIONS AND TREATMENT AT GUNTERSVILLE DAM

By Verne Gongwer,[3] M. Am. Soc. C. E.

Synopsis

Foundation conditions at Guntersville Dam presented interesting engineering and construction problems, the solutions of which are thought to be unique. Since techniques for foundation treatment were developed progressively to suit the varying conditions encountered, the different parts of the foundations are described in the order in which they were uncovered. Ordinary procedures, conditions, and methods are described only briefly to permit a more complete description of extraordinary conditions and ground-water behavior after the filling of the reservoir.

General Features of the Project

Guntersville Dam consists of a reinforced concrete spillway structure 856.0 ft long in the river channel. On the right or north bank there is a navigation lock, with a chamber 60 ft by 360 ft; and on the left or south bank is the power house with three 24,300-kw generating units, space being provided for a fourth unit when it is required (Fig. 31).

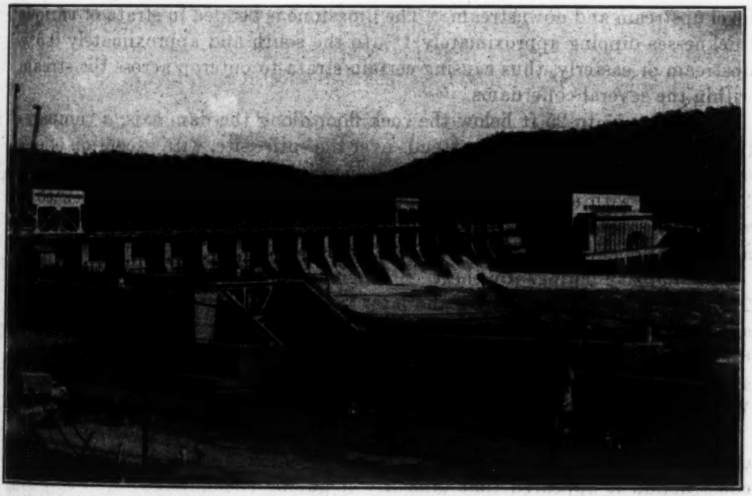

Fig. 31.—Guntersville Dam, Showing Lock, Spillway, Power House, and Switchyard

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[3] Tacoma, Wash.; formerly Project Engr., Guntersville Dam, TVA, Guntersville Dam, Ala.

The rolled earth-fill ends of the dam abut against limestone bluffs about 4,000 ft apart at pool level. The left abutment is quite precipitous and regular from the rock floor of the valley to pool level, whereas the right bluff, from the rock floor to pool level, consists of overhanging limestone strata, rendered exceedingly ragged by the processes of solution, remarkably eroded, and overlain generally with clay 4 to 6 ft deep. The lock, spillway, and power-house structures lie nearer to the right bluff than the left, and the remainder of the dam consists of rolled earth dikes, approximately 800 ft long on the north and 1,600 ft long on the south, with upstream and downstream slopes of 1 on 3, and heights of from 35 ft to 45 ft.

In general design, Guntersville Dam is somewhat similar to Chickamauga Dam and Pickwick Landing Dam, but the foundation conditions are dissimilar to a large extent.

From a construction viewpoint, the site of the dam is in a deep deposit of Bangor limestone, in which the river has eroded a wide, comparatively shallow canyon, with a fairly level rock floor. Upon this floor silt and alluvium average about 35 ft deep. The materials are arranged in general with quite permeable gravel, from 0 to 5 ft thick, lying directly upon the rock, and fine gravel, coarse sand, fine sand, sandy silt, and clay (in the order named) from the bottom to the top of the flood plain. The stream bed, approximately 1,100 ft wide at the dam site, has been swept clean down to the rock in some places.

The rock floor has an inclination of about 1% to the south and is practically level upstream and downstream. The limestone is bedded in strata of various thicknesses dipping approximately 1% to the south and approximately 0.5% upstream or easterly, thus causing certain strata to outcrop across the stream within the several cofferdams.

Roughly, 15 to 25 ft below the rock floor along the dam axis, a comparatively thin double-shale seam extends over the entire site, with exception of one or two small areas. The shale seam, generally, is in two members from 6 to 12 in. thick with a limestone member between, ranging from 2 ft thick to 0, where the shale members blend into one, and even disappear over the small areas mentioned.

Preliminary borings (by the U. S. Engineer Department), generally 100 to 250 ft apart, in several ranges, and later borings, indicated general conditions and indicated the selection of this particular site as compared with others near the head of Wheeler Pool. The location of Wheeler Dam (Fig. 22), and the approximate pool elevation, had previously been fixed by the construction of Wheeler navigation lock by the U. S. Engineers.

The preliminary borings, however, did not reveal the peculiar detail features of the rock, which later presented several construction difficulties. During early construction it appeared that difficulty might be encountered for the entire length of the dam. However, as the work progressed advantage could be taken of certain favorable conditions that resulted in ultimate costs well within the preliminary estimates. Originally, only the intakes for a future power house were contemplated and approved for construction, no generating units being authorized. Circular-cell, steel sheet piling cofferdams were adopted to exclude a maximum height of water of approximately 40 ft above the top of rock. This river stage should be expected about every three years, but was nearly reached during three of the four ensuing winters.

Permeable gravel samples from certain test pits, the comparatively high specific capacity of some of the pits under pumping tests, and the fairly smooth rock floor indicated by the preliminary borings resulted in an early decision to drive a steel sheet pile cutoff to rock across the entire width of the flood plains.

STEEL PILE CUTOFF UNDER NORTH EMBANKMENT

From the lock wall nearly to the north abutment, the sheet piling was driven to a very uniform rock surface. However, near the north abutment the piling (see Fig. 32) ran up over several large residual boulders embedded in residual clay, with a thin edge of the pervious gravel layer beneath. These boulders were removed and the piles were trimmed and set down upon the smooth rock floor beneath. The backfilling of this excavation was compacted with air tampers, thus insuring practically ideal compaction. The sealing of the piles to the rock floor was otherwise considered satisfactory, and the seal to the rock escarpment was obtained by means of a concrete cutoff wall.

LOCK FOUNDATION

The lock cofferdam was unwatered and cleared of mud, gravel, and loose boulders in moderate number and size. The hydraulic dredge was only partly successful because of boulders and eroded rock surface. Cleaning of an area for a pump sump revealed enlarged joint cracks at approximately right angles, dividing the top stratum of rock into nearly rectangular blocks (Fig. 33). As

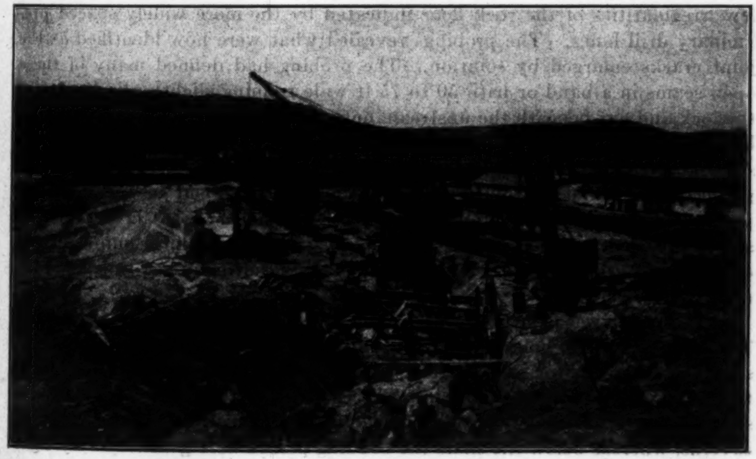

FIG. 32.—STEEL PILE CUTOFF UNDER NORTH EMBANKMENT, SHOWING TRENCH FOR CUTOFF WALL

the sump excavation progressed, the bedding planes and joint cracks were found to be peculiarly enlarged and distorted by solution (that is "solutionized"), the joint cracks usually being narrow at the top, widening downward,

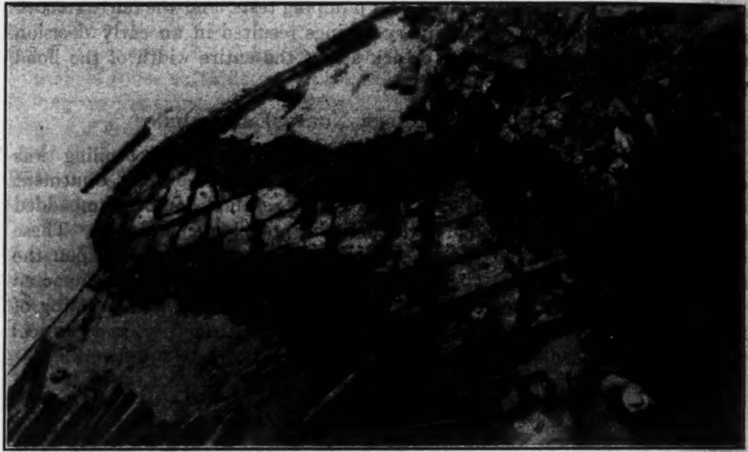

FIG. 33.—JOINTS AND SOLUTION CHANNELS IN THE TOP STRATUM OF ROCK IN THE LOCK COFFERDAM

and staggered somewhat in the different strata. This condition continued downward to the general shale seam.

Previously probings were made from a barge in an attempt to explain a few irregularities of the rock floor indicated by the more widely spaced preliminary drill holes. The probings revealed what were now identified as the joint cracks enlarged by solution. The probing had defined many of these open seams in a band or path 50 to 75 ft wide running slightly diagonal with the lock and out beneath the upstream outer corner of the cofferdam. At the latter place leaks of considerable magnitude developed early (see Fig. 34) since many "solutionized" bedding planes were filled only partly with quite permeable material. "Solutionizing" near the upper end of the lock had progressed so far that the upper strata had subsided 2 or 3 ft in some places and were absent at other points. The excavating and hauling equipment frequently broke through these undermined layers, resulting in lost time and high repair costs.

Exploration was expedited with wagon drills and small and large shot drills. The 36-in. shot-drill holes, at frequent intervals, were a speedy and especially satisfactory means for examination. It was soon found that in areas where the action of solution was most pronounced, most of the rock above the shale seam was questionable, with solution channels occasionally amounting to small caverns, whereas below the shale the bedding planes were generally very tight and vertical joints few and equally tight.

Due to the slight inclination of the strata, the shale rose to within about 3 ft of the rock surface near the downstream end of the lock. The excavation, starting there, was close-drilled to concrete lines and extended to the bottom

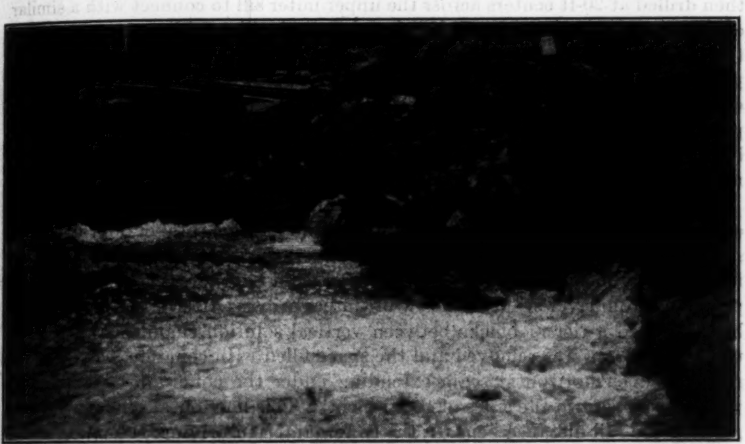

FIG. 34.—LEAKS THROUGH SOLUTION CHANNELS BENEATH UPPER END OF LOCK COFFERDAM

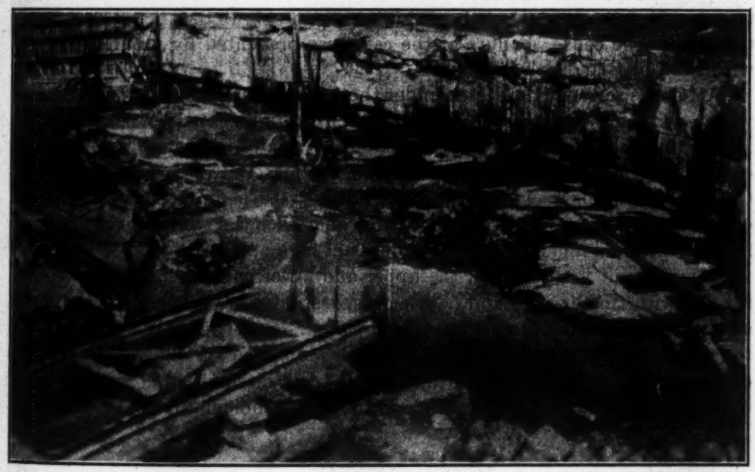

FIG. 35.—LOCK WALL FOUNDATION, SHOWING SHALE SEAM NEAR THE BOTTOM OF THE EXCAVATION

of the shale seam (which can be seen in Fig. 35). As the shale dipped deeper upstream an attempt was made to save a local area of quite sound rock. A fairly heavy blast disposed of the matter by shifting a portion of this rock along the shale about 3 in., as revealed in one of the 36-in. holes. The entire lock excavation, therefore, was extended to the bottom of the shale.

The remaining minor seams in the rock below the lock foundations were treated by drilling low-pressure holes on 20-ft by 17-ft centers and grouting at 30 lb per sq in. before placing concrete. A curtain of high-pressure holes was then drilled at 20-ft centers across the upper miter sill to connect with a similar curtain under the spillway and under the north dike. Pipes were extended through the concrete and these holes were later grouted at a pressure of 60 lb per sq in. Seepage into the excavation from below the shale was negligible, even before grouting, and little grout was taken by either low-pressure or high-pressure holes. A sound area of rock above the shale landward of the lock furnished a tight foundation and a connection for the steel pile cutoff of the north dike.

During operations in the lock cofferdam, several serious leaks or blows occurred, filling the cofferdam several times and losing a total of approximately six weeks' progress. Temporary closures were made several times from the river side by means of many barge loads of sand bags, gravel, and baled hay, protected from river scour by a row of timber cribs. After one such blow had been stopped a tier of blocks between vertical solution channels and parallel to the cofferdam was removed and the space filled with concrete. (In Fig. 36, note the open horizontal seam extending under the cofferdam. Convenient vertical seams form the sides of the trench.) This horizontal solution channel reached the full distance beneath the cofferdam. In driving, the piling of the cells had seated firmly upon the level upper rock stratum, causing the tops of the piles to appear very uniform and satisfactory. At several points "chimneys" in the rock sucked in the cell filling as the blows began and threatened to collapse the cells. Failure of the cells was prevented by prompt plugging with sand bags and baled hay.

Attempts to grout the seams under the upper end of the cofferdam through wagon-drill holes immediately inside the cells were defeated by flowing water. Blanketing these particular leaks on the river side of the cofferdam by means of hay, gravel, sand bags, etc., met with no material or permanent results. Outside blanketing was unduly expensive, since it was impossible to locate all of the numerous entrances to the seams, and since the work required a large force of men and much floating equipment of high operating cost. As the lock walls were built high enough, the upper and lower ends of the lock were barricaded with timber cribs and needle dam, respectively, to allow the lock floor and other work in the chamber to be completed. Stoppage of leaks by grouting the open seams through a line of drill holes along and through the center line of the cells with the cofferdam full of water was discussed; but the necessary delay was deemed too great, and the results too problematical.

SPILLWAY FOUNDATION

After the second cofferdam had been started out into the river, it was decided to drill through the center of the cells on 10-ft centers and grout all seams in the rock and the gravel filling of seams under and between residual boulders. A second row of holes was added, staggered with the first, except for the downstream arm of the cofferdam. Here conditions appeared to be better, and 20-ft spacing was adopted. The holes were drilled through the shale a few feet. The ragged profile of the tops of the piles in certain locations, as seen in Fig. 37, indicated groups of residual boulders at the bottom of the piling. Smooth appearing piles were found frequently over badly undermined layers.

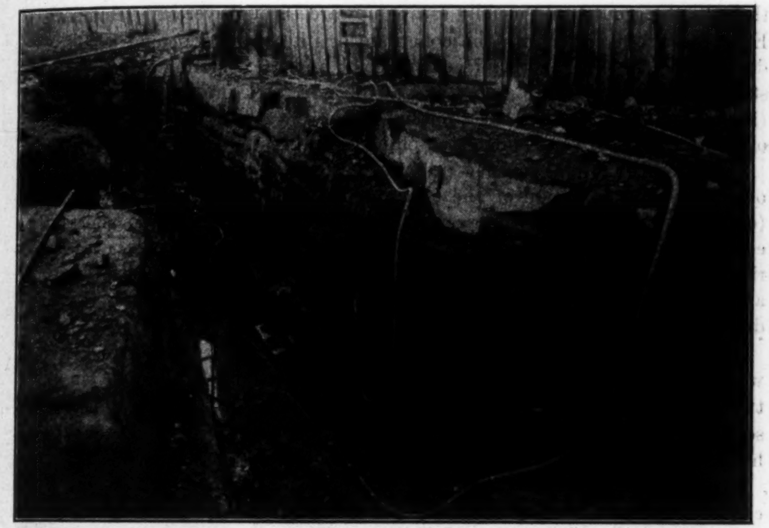

FIG. 36.—TRENCH FOR SEALING HORIZONTAL SOLUTION CHANNELS THAT HAS PERMITTED COFFERDAMS TO FILL

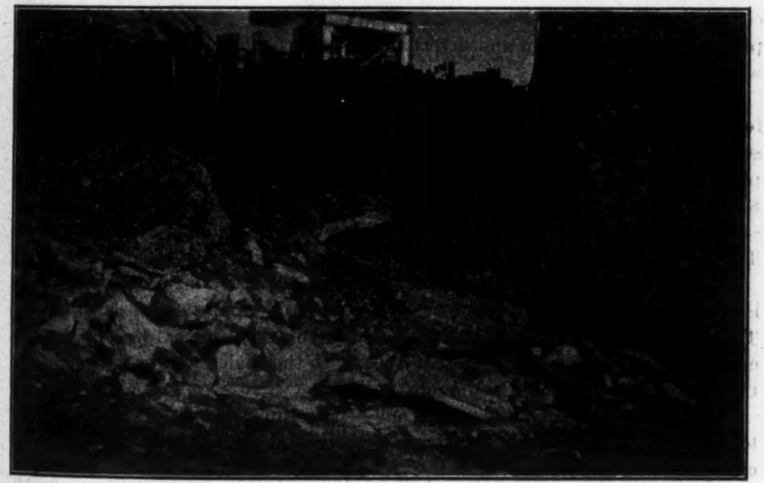

FIG. 37.—CELL FILLING IMPREGNATED BY CEMENT GROUT ESCAPING FROM BELOW

The grout holes were located along the center line of the cells in order to render the grouting most effective by insuring that, whichever way the grout traveled from the hole, the pervious seam-filling material or open seams would be solidified for a distance equal at least to one half the width of the cells. Outside of the cells the grout could usually escape upward through vertical "chimneys." Escape of grout either into the cofferdam or into the river could usually be detected by bubbles, froth, or cement color and the grouting was slowed or stopped until it began to stiffen and a sufficient back pressure was indicated (by gages on the grout lines). Special holes were drilled and grouted slowly opposite the "doorways" under the piling caused by boulders.

The resulting total leakage into this cofferdam was negligible, requiring only part-time operation of one 6-in. pump; yet the head on the cofferdam (caused by filling Wheeler Reservoir a short time previously) was constantly equal to (and greater than, during a July flood) the head produced by occasional river rise, which had invariably caused the lock cofferdam to blow. Grouting a few nominal leaks against full cofferdam head (with cement, sand, and sawdust mixtures) met with little success.

As the excavation progressed, it was found that the particular rock strata which gave trouble in cofferdam No. 1 were generally quite sound, but that two or three superimposed strata existed which were extensively affected by solution. Had the grouting not prevented, this condition obviously would have permitted quite as troublesome leaks and blows as in cofferdam No. 1.

Construction in cofferdam No. 2, due to the resulting dry and favorable conditions, was completed in approximately four months, obviating expensive delays and disruption of the construction schedule. The direct cost of drilling and grouting was substantially less than the various and repeated remedial measures necessary in cofferdam No. 1. Also it was imperative that work be completed in cofferdam No. 2 in this remarkably short interval in order to complete the third cofferdam and divert the river through the spillway before the winter floods.

Detailed exploration of the rock in cofferdam No. 2 revealed that, except for the first spillway pier south of the lock, there was a depth of 16 to 18 ft of sound rock (Fig. 38) above the generally occurring thin shale strata; and the plans were revised to set the spillway on this rock, thus obviating an estimated $300,000 worth of line drilling, rock excavation, and reinforced concrete necessary to carry the piers and also the weirs down to the stratum below the shale. This also saved much valuable time and indirect cost.

When cleaned off, the river-bed rock was practically level and characteristically scoured by river action, with grooves and ripples as seen in Fig. 39. There were a number of tight vertical-joint cracks, and a few which were enlarged by solution ("solutionized") nearly to the shale. These were cleaned thoroughly and sealed with concrete.

It was also necessary to remove one or two top strata for a short distance to eliminate the extensive partly open horizontal seam which had been the cause of much trouble in cofferdam No. 1. In this seam were found quantities of the characteristically colored sand and clay used in blanketing operations for cofferdam No. 1. As this general seam also ran beneath the upstream arm of cofferdam No. 2 it had evidently been sealed thoroughly this time, by the grouting operations.

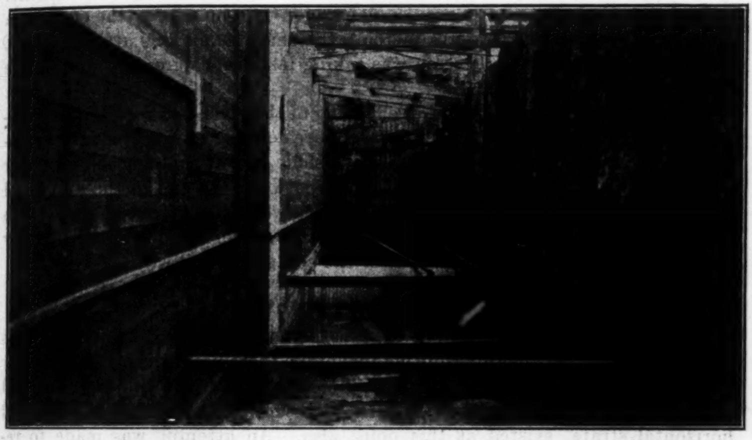

FIG. 38.—EXCAVATION FOR SPILLWAY PIER NO. 2, SHOWING SHALE SEAM AND SOUND ROCK ABOVE

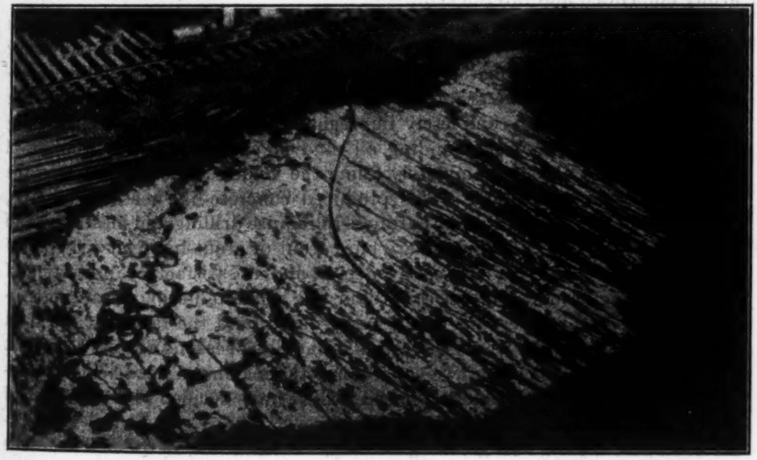

FIG. 39.—APPEARANCE OF RIVER BOTTOM IN A PART OF SPILLWAY COFFERDAM NO. 2

As a basis for the decision to save the sound strata above the shale, 36-in. drill holes for inspection were sunk to, and through, the shale at the upstream and downstream ends of each spillway pier and also on many joint cracks.

The shale here was found to be relatively hard and intact. Before filling the holes with concrete, baskets of heavy steel reinforcement were inserted. Grout at 30 lb of pressure in holes piercing the shale seams 3 ft horizontally from the vertical rock face at pier No. 2 (see Fig. 38) did not appear at the cut rock face. As a precautionary measure, grouting of wagon-drill holes over the entire area of the spillway on 20-ft squares piercing the shale seams was attempted, but was abandoned as the shale seam was taking insufficient grout to warrant the operation.

As a precaution against erosion or lifting of the river-bed rock strata downstream from the spillway apron, 1.25-in. square dowels were grouted into wagon-drill holes extending several feet below the shale on 5-ft centers each way, over the entire exposed area, about 75 ft wide, between the end of the apron and the cofferdam. Open-joint cracks in this area were cleaned out and filled with concrete. Subsequent soundings indicated that no rock had been displaced by the succeeding heavy winter flood over the completed spillway.

Power-House Foundation

On account of the exceeding dryness of cofferdam No. 2, question was raised as to whether the full grouting technique there used was necessary under the third-stage cofferdam. The driving of a few upstream cells corroborated the expectation that residual boulders, representing the remnants of several horizontal strata, existed at this point also. An attempt was made to remove the boulders with a clamshell bucket, but they proved too large and too firmly embedded and interlocked.

One minor drawback connected with grouting of the second cofferdam was already apparent, in that some difficulty was being experienced in redredging the hydraulic cell-filling material from cofferdam No. 2, for use in filling cofferdam No. 3. Some of the grout had escaped upward into the hydraulic filling in certain cells, almost completely impregnating all pervious gravel, pushing aside, compacting, and surrounding all mud or silt contained in the gravel where it was not already completely filling the voids sufficiently to render the gravel impervious to grout. This produced concrete-like masses (Fig. 37), many of which were of sufficient hardness to require drilling and blasting. (All lumps and masses seen in Fig. 37 are pure grout or grout-impregnated gravel.)

However, considering the costly delays and unsatisfactory protection of the blanketing around the first cofferdam, the rapid and economical construction in the second cofferdam which was grouted, and the fact that the third cofferdam must survive two winter and spring flood seasons against somewhat higher heads than the other two cofferdams (the maximum head being approximately 45 ft above the top of river-bed rock), it was decided to grout the third cofferdam in a similar manner.

Since the drilling and grouting had lagged somewhat, in order that results of driving of some of the cells could be observed first, it was finally decided to attempt unwatering while there still remained a portion of the lower arm, nearest the river bank, which had not been grouted. Upon unwatering, the cofferdam held satisfactorily for several weeks, and construction work was fairly well under way, during which time the grouting of the remaining cells under the unbalanced head proceeded with no definite results. Two or three incipient blows then occurred, one of which washed away the gravel berm against the inside of one cell. It was stopped by dropping sand bags and hay into the caving cell filling, and was definitely cured by drilling several extra grout holes. Another leak consisted of several mud boils, with a small flow of water, at the still ungrouted land end of the downstream arm and which, peculiarly, stopped completely of its own accord in about 30 min. About two weeks later, however, and before the grouting was completed near this point, the same boils reopened in earnest and within a short time the blows had increased until all equipment had to be removed from the cofferdam, and it was allowed to fill with water. Approximately 2.5 weeks of construction were lost while completing, under balanced head, the grouting of the land end of the cofferdam and for some distance into and upstream along the river bank where part of the blow occurred. When pumped out again, this corner of the cofferdam was perfectly tight and remained so. None of the minor leaks increased, in the upper arm or elsewhere, where grouting had been completed before unwatering. The general statement can be made, therefore, that at no point in either the second or third cofferdam were any but very minor leaks subsequently experienced where grouting had been completed under balanced-head conditions.

It was quite important to trace the flow of grout in certain holes that failed to build up pressure. Just prior to the blow under the downstream arm fluorescein dye was introduced into the holes. Quite soon it was detected emerging with clear water that had been flowing from open grout pipes at the bottom of the river end of the south steel pile cutoff wall, near its connection with the power house. The dye had traveled approximately 700 ft and presumably followed the same seams or "solutionized" bedding planes in the rock which a short time later caused the cofferdam to blow. The direction followed by the dye corresponded to one of the two jointing axes of the vicinity. These incidents illustrate the nature and condition of the upper rock strata, as well as the effectiveness of the grouting under these conditions, and they may suggest many desirable applications of this process.

Unwatering of the third cofferdam revealed several upper strata entirely reduced by solution to residual boulders in place and covering most of the power-house and tailrace location. Very few of the residual boulders have apparently moved from their original positions in the strata (see Fig. 28 submitted by Mr. Ross). The removal of the boulders revealed generally sound rock in the same stratum upon which the spillway had been built. No part of this stratum was undermined by open horizontal seams.

The remaining three spillway bays and the power-house intakes were founded upon this layer, as in the second cofferdam, and with similar preparation and cleaning of the rock. The excavation for the draft tubes of the generating units extended some distance below the previously described shale seam, beneath which the bedding was tight and was water bearing only to a negligible extent under the approximately 95-ft head existing at the bottom of several 36-in. drill holes.

Water testing proved all exploration holes to be remarkably tight, subsequently taking very little grout. The general nature of the power-house

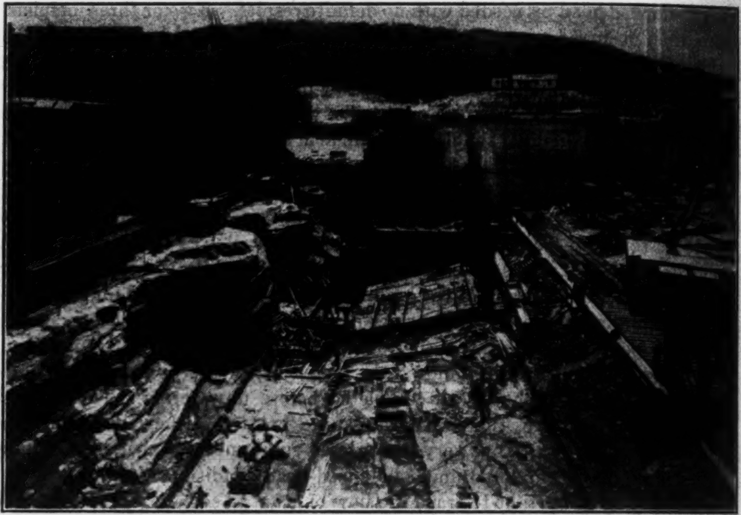

Fig. 40.—Facing South from the South Spillway Training Wall, Showing Power-House Excavation

foundations is seen in Fig. 40. The double shale seam is readily discernible. A complete pattern of holes was drilled and grouted under the turbine placements and entire intake to insure maximum bearing and to block off any small seams.

Grout Curtain

A grout curtain consisting of high-pressure and low-pressure grout holes was extended across under the entire upstream face of the dam, including the intake, spillway, and upper miter sill of the lock. The low-pressure holes were drilled by wagon drill to a point below the shale, in two rows. One row of holes was drilled at approximately 15° inclination from vertical in one direction parallel to the axis and at 5-ft centers, whereas the second row was 2 ft downstream, spaced at 10-ft centers and equally inclined in the opposite direction, forming a pattern by which any vertical joint crack existing between the shale and the surface would be pierced in one or more places by grout holes. The open seams encountered were predominantly horizontal bedding planes. The low-pressure holes or grout curtain were grouted at pressures of approximately 25 lb per sq in. before concrete was placed. Displacement gages and a wye level were used at numerous points to detect any possible heaving of the rock strata. No displacement was detected at any point, although a very slight displacement probably occurred in one spillway bay where grout from high-pressure holes was found emerging from certain dowel holes and seams in the rock downstream from the spillway apron. Subsequently, a 36-in. drill hole immediately below the spillway baffle disclosed a layer of grout less than $\frac{1}{16}$ in. thick on top of one of the shale members.

The high-pressure holes were drilled through previously embedded pipes after the spillway weirs and floor of the intakes were placed, and were grouted at pressures of 50 to 60 lb per sq in. The quantity of grout taken by both low-pressure and high-pressure holes was very nominal except in isolated cases where there were very small open seams. Drain holes downstream from the grout curtain and under the spillway apron and draft tubes were drilled after all grouting was completed.

Steel Sheet Pile Cutoff Under South Earth Embankment

Investigation and correction of seams and caverns in the rock at the bottom of the steel pile cutoff under the south earth dike developed into a major operation. A temporary row of steel sheet piles was driven about 12 ft upstream from the cutoff piling, forming a sheathed trench as shown in Fig. 41.

The first section of the trench was opened at a typical location where the piles had seated very irregularly. As soon as the pervious gravel strata were reached, water came in copiously, and when the excavation encountered the rock the remnants of one or two rock strata were disclosed which had been "solutionized" into nests of residual boulders with the spaces between filled mostly with very pervious water-bearing gravel (see Fig. 42). After removal of these boulders, the surface of the next stratum was found to be quite regular and relatively free from cracks, and the piles were reseated tightly on it after trimming off bent and injured ends. Jackhammer exploration through this stratum disclosed an open seam 4 in. to 6 in. deep, 3 or 4 ft beneath the surface. A trench was opened down to this seam, and jackhammer holes extending 10 to 12 ft deeper revealed no further seams. Furthermore, no seams were revealed by the deep shot-drill exploration holes, which had been increased in number until by that time, with the original prospecting holes, there were deep shot-drill holes approximately 100 ft apart. The trench in the rock was then filled with concrete to seal the seam, and a concrete toe wall was cast along the upstream side

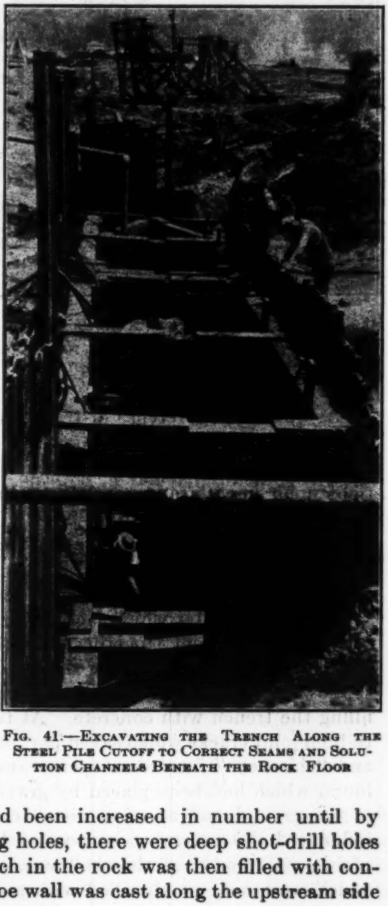

Fig. 41.—Excavating the Trench Along the Steel Pile Cutoff to Correct Seams and Solution Channels Beneath the Rock Floor

of the piling about 3 ft high, except where required to be higher to cover split interlocks of the piling. The trench between the sheet piling was backfilled with puddled clay and silt for a number of feet, above which silt and fine sand were used. The temporary piling was finally pulled and reused. Small shot drills were used constantly along the axis of the cutoff in seeking out additional information.

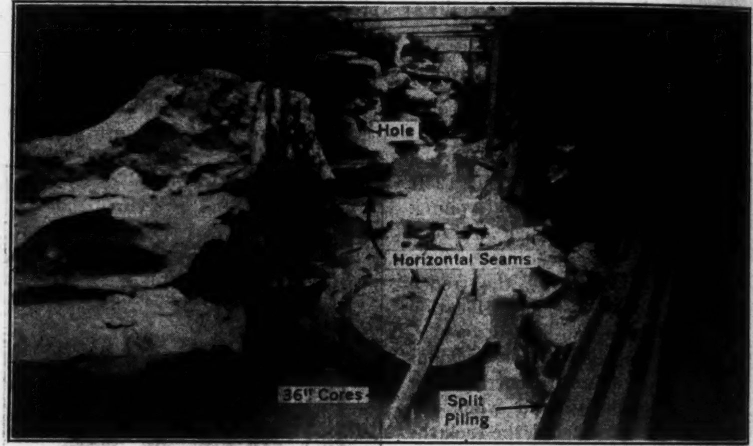

Fig. 42.—Typical Condition at the Bottom of a Cutoff, Showing Nests of Residual Boulders

As additional stretches of trench were opened, where the piling was most uneven, a variety of conditions was found. At many places where the piling tops were smooth and regular, and the bottoms were seated square and watertight upon smooth, apparently sound rock, the worst seams were often found beneath, at depths from 10 to 25 ft. This caused the trench to be opened for the entire length of the cutoff, and a technique was developed, which resulted in greater progress and economy.

In the various sections of the trench there were usually residual boulders to be removed, after which jackhammer, wagon-drill and deep shot-drill holes were sunk, sufficient (together with the preliminary holes) to determine the existence and depth of any open horizontal seams. Horizontal seams at shallow depths were corrected by close drilling and trenching, and later backfilling the trench with concrete. At times the top stratum would have longitudinal joint cracks which saved part of the drilling, and transverse joint cracks and "chimneys" into some open seam below. In one chimney concrete was found which had been placed by gravity through 6-in. pipes from the surface of the ground and at some distance away, horizontally. Such "chimneys" influenced the general ground-water behavior and invited sinkholes in flood plains. A number of the shallower seams opened into seams 2 or 3 ft deep which were only partly filled with sand and gravel. As pumping in sumps continued, the general water table would soon drop until the water no longer came into the trench at the top of rock. Due to "chimneys" and vertical seams outside of the trench, it would run in freely through the lower horizontal seams, to the capacity of one or two 10-in. pumps. All manner of weird variations of structure and drainage were found.

Where it was not economical or feasible to open-cut the rock down to the seams, they were entered from the end in some instances, or through a series of 36-in. drill holes, and cleaned of all gravel, sand, and mud. Tight barricades consisting of bags of concrete were placed in the seams upstream and downstream, beneath and parallel to the steel pile cutoff, with 6-ft or 8-ft space between the barricades. The rock floor and roof were cleaned, and roughened if necessary, and a permanent concrete plug or wall was placed through the 36-in. drill holes. Check holes were afterward drilled through the roof and plug, and tested, to insure that the seam was tightly filled.

The number of deep 3.5-in. shot-drill holes was increased progressively until finally they were from 12 to 16 ft apart throughout the length of the entire cutoff. They reached well beneath the shale, and frequently much deeper.

The large seams were always found above the shale, although at two or three places the faulty condition extended to, and included, the shale. One such place, when cleaned out, developed into a cave approximately 10 ft high, under and roughly parallel to the cutoff, with about 20 ft of sound rock overhead. This cave had numerous branching arms and connecting seams. Figs. 43 and 44 are typical of the many horizontal seams encountered, which were usually of considerable extent, both longitudinally and laterally to the cutoff wall, having random branch caves and openings of unknown extent. In Fig. 44, light reflections indicate an adjoining 36-in. hole. Five men lying prone, end to end, passed bags of concrete to build forms for a plug in Fig. 43. Fluorescein dye introduced into ground-water observation wells of perforated pipe beyond the upstream and downstream toes of the earth dam, and into several small sinkholes at the left abutment, appeared in the pump sumps within a few minutes by way of such typical seams as those illustrated.

For some distance along the cutoff there were two—and even three—seams, each 2 to 3 ft high, one below the other, the lowest one being about 25 ft below the top of rock. Workmen entered the seams by 36-in. drill holes (approximately 12 ft apart) to clean them out and place the barricades for confining the concrete plugs. An interesting game ensued in placing the concrete of the plugs and backing out with the pumps in such a manner that all concrete was placed "in the dry" and had opportunity to set without damage by flowing water.

Since a difference of head of 3 or 4 ft had developed above and below the cutoff due to diversion of the river over the spillway and the sealing of all other communicating seams conducting ground water, a considerable cross flow was expected at the point of final closure which was to be the pinching out end of the cave previously mentioned. The cave itself had been treated by open-cut methods from an adjacent open pit. To close the remaining 25-ft gap of low seam successive and interlocking holes were drilled 36 in. in diameter, allowing the ground water to rise and practically come to rest before filling each hole full of tremie concrete, then pumping down again and drilling the next hole so that a segment of the concrete filling of the last hole was obtained in each succeeding core. This was a slow but necessary process as the concrete apparently did not spread out in the seam very far from the bottom of each hole, or else it was washed away. An effective practice in this operation was to drill the 36-in. holes 3 or 4 ft below the seam to form a catch basin for sand and gravel, as well as a pump sump, thus assuring a full and solid concrete section

FIG. 43.—HORIZONTAL SEAM OF CONSIDERABLE EXTENT

FIG. 44.—TYPICAL SEAM SHOWING THE CONCRETE BAGS IN PLACE FOR FORMING THE CONCRETE PLUG UNDER THE CUTOFF

across the seam. Check holes were then drilled with a smaller core drill in the intersegments of the 36-in. holes to insure soundness and watertightness in the concrete filling. An expanding steel cylinder and other equipment and ideas had been devised, such as the possible use of 36-in., very thin, metal concrete buckets, or the use of a rock core lowered in heavy canvas sack later to be filled with grout, in case the method had not been successful.

As a final operation in all sections of the trenches, wagon-drill holes were sunk, 30 ft deep on 24-in. centers, along practically the entire line of the cutoff to avoid overlooking small local seams or defects. These holes, the 3.5-in. shot-drill holes on 12-ft to 18-ft centers, and jackhammer holes were all fitted with pipes and grouted to refusal in order to seal the small horizontal seams, both above and below the shale stratum. At the pressures used (40 to 50 lb per sq in.) the neat-cement grout traveled considerable distances in very small seams throughout various parts of the work. Any horizontal seams small or large, at bedding planes, usually could be depended upon to show up in several adjacent holes, and it was nearly impossible for them to escape correction. Certain "solutionized" vertical jointing seams, not definitely sealed otherwise, were cut off by concrete-filled 36-in. drill holes. The predominant open seams, however, were horizontal, and open vertical seams were seldom encountered except where they were visible in the topmost stratum.

Guided by the special experience and detailed knowledge of conditions gained in successive trenches, as a precautionary measure the first trench completed was subsequently drilled on 5-ft centers to a depth well below the shale seams, and regrouted.

Except for final grouting and backfilling, the last operation in all trenches was the construction of the concrete sealing wall along the bottom of the piling, filling all trenches and large drill holes, and sealing all splits in the piling interlocks by carrying the wall upward as necessary.

A flashlight beam could be thrown for considerable distance in some of the seams and the system of seams was proved (by fluorescein) to extend entirely through under the earth-dam section. Therefore after filling the reservoir there was a possibility that sinkholes might be formed in the flood plain or in the toe or slope of the earth dam by flood-plain material being carried down through "chimneys" into voids in the seams although all seams were completely blocked at the cutoff as herein described. To obviate this possibility, a system of holes was drilled both upstream and downstream, into, and in the vicinity of, the known open seams, and all voids that were found were filled by grouting with a mixture of fine flood-plain sand mixed with cement in proportions of approximately 1 to 4. This mixture was found to travel well and to be capable of setting quite firmly under water. During the time that some of the trenches were open, considerable ground was lost into the trench, at places, due to heavy rains and also to movement of ground water. These losses occurred under the sheet piling at boulders and split interlocks, and into seams and cavities outside of the trench, resulting in some open holes to the surface. To guard against voids, a water jet was used to feel out all voids which did not run to the surface, and fine sandy silt was added and jetted into place to refusal. The jetting was done carefully as the puddled backfilling was placed in the trench in such manner as not to displace the piling in either direction, although very slight displacement did occur at two or three points, in an upstream direction.

As the work progressed and plottings of the original exploratory borings were studied with respect to the unexpected conditions found (which required the extensive treatment described herein), it was seen that preliminary holes 100 to 200 ft apart were entirely inadequate in soluble limestone as a basis for proper deductions and design. The horizontal extent of the seams is not made apparent; and only an occasional hole, if any, will give any previous hint of the worst conditions to be encountered. The logs of these holes can be diagnosed properly only with the help of additional adjacent holes. In regions of soluble limestone, the preliminary holes, for proper design and estimating (although seemingly excessive cost and time are involved), should be not more than 25 ft apart along the axis expected to be adopted and many of the holes should be very deep ones. All holes showing open or mud-filled seams, appreciable loss of core, or loss of much water under test or during drilling, should be supplemented by adjacent holes at 5-ft to 10-ft spacing for the purpose of delimiting the extent of such seams in all directions.

The equipment usually available or supplied for sinking test pits, and for pump testing therein, is often very inadequate for the purpose of obtaining full and essential information. It is believed that, where preliminary borings show defects in the rock, one of two procedures is indicated: Either (1) fully adequate equipment and methods should be used for sinking test pits, safely and satisfactorily; or (2) casings, at least 36 in. in diameter, should be driven to rock, with large holes drilled in them and with pumps of the proper type and sufficient capacity to permit examination and to determine the full water-producing capacity of such seams. The writer believes that fluorescein tests should also be made, from well points or other test pits some distance away. A variation that is sometimes effective is to fill certain pits or pipes with water and observe the amount and time interval of effect on water levels in adjacent pits and wells. Added information may often be had by use of fluorescein, sal-ammoniac, or other chemicals. Most satisfactory, expeditious, and economical results can be obtained under adverse conditions by a 36-in. drill, or larger, sinking a casing through the flood plain, and drilling into the rock well below all pertinent seams to form an ample catch basin for sand and gravel during pumping tests and to permit entrance and inspection of the lowest seams. One such 36-in. hole was sunk through the flood plain and about 50 ft into the rock on the center line of one of the generating units; and another such hole was drilled through a crib set in the river. The pumping will frequently require a 10-in. well-type pump, or larger, and it will usually promote speedy results and economy in the end to employ equipment capable of coping fully with conditions.

With full information available in advance as to the exact conditions which would be encountered in perfecting a cutoff at this site, there would still doubtless have been differences of opinion as to whether the work should be prosecuted by open-cut or by sheathed trench methods. It is the opinion of the management on this job, considering the particular conditions existing, that the method used was somewhat more economical, and quicker, and fully as certain of positive results as if an open-cut had been made to rock.

Foundation of Rolled-Fill Sections

Design specifications required that the top of the steel pile cutoff should extend a minimum of 5 ft into the rolled-fill embankments. Comparative estimates indicated the economy of selecting the pile lengths so that they would drive approximately flush with the original ground after the stripping had been done. A cutoff trench 5 ft deep and approximately 12 ft wide on each side of the piling was excavated and the rolled fills were started by compacting the backfilling material of these trenches with sheepsfoot rollers, under laboratory control. In this way it was possible to reach soil of maximum natural compaction, and was considered preferable to shorter piling and a deeper and narrower trench with puddled backfilling. The most homogeneous results were obtained by making these trenches of liberal width.

It was found necessary to puddle the deep, narrow sheathed trench which had subsequently been excavated to the bottom of the cutoff piling for correction of boulders, cavities, and seams. However, the material used for puddle (except near the bottom of the piling) permitted fairly free draining and had considerable time to settle and shrink before the rolled fill was started.

Three years of observations on the rolled fills indicate that settlement had substantially ceased early in 1940, with a maximum of approximately 5 in. and 4 in. in the approximate 70 ft from the top of the rock floor to the crest of the dam, for the north and south fills respectively, there being about one third of the settlement in the 35 ft of rolled fill, and approximately two thirds in the 35 ft of underlying flood-plain materials. The results were approximately the same for both the south embankment, with the 12-ft wide puddle trench to the rock floor just upstream from the cutoff piling, and for the north side where such a trench was not necessary.

The rolled materials varied from the finest of river silt and sandy silt to residual clay, the latter being used for the plug next to the lock wall. Here the total settlement is approximately $1\frac{3}{4}$ in. for a wedge of rolled residual clay 70 ft high resting on the rock on its lower edge, which is about 20 ft thick, the other supporting materials being the lock excavation slope of fine river silt and the end slope of a section of rolled embankment of fine silt. However, on the south side of the river, for a switchyard electrical cable tunnel resting on a similar plug of sandier river silt adjoining, but not a part of the dam, there has been a maximum total settlement of approximately 4 in.

Efficacy of Cutoff Provisions and Corrective Measures Applied to Seams and Cavities, as Indicated by Ground-Water Observations and Other Evidence

Since the pool was raised to reach normal operating levels in the latter part of January, 1939, the behavior of the ground water downstream of the north and south embankments has been studied by means of a number of observation wells. Time graphs have been made of the periodic observations, and ground-water contour maps have been prepared at intervals of 5 to 7 days.

On the north side of the river, a general seam of open gravel 3 or 4 ft thick was exposed on top of the rock in the lock excavation and from evidence of pits and drill holes the gravel tapers out irregularly near the north abutment. The wells terminating in the gravel respond to tailwater variations in proportion to their distance from the river; a few, in more impervious materials, respond to rainfall; and none respond noticeably to ordinary pool-level variations. One well pipe near an original spring had been flowing for about two years. When the pool was filled it was decided to put a riser on this pipe, and the water level in it rose about 10 ft above the ground surface. The head on this well had not been determined previously. This effect (after much study and the addition of several wells, chemical analyses of water, temperature observations, etc.) has been proved to be purely artesian and sensitive only to rainfall, which probably percolates down the side of the mountain and produces pressure in seams in the rock, with which the one particular well came in contact. All observation well casings were perforated for 5 ft at the bottom and driven to rock.

Slow saturation of the north abutment apparently tilted and somewhat changed the original direction of ground-water flow. Possibly this change augmented the flow slightly, as the original spring mentioned, plus flow from the artesian pipe, increased by increments following certain rains, from a total of 5 gal per min to 25 gal per min over a period of 3 months and has held practically steady, at 24 gal per min, ever since.

On the south flood plain, at the beginning of the work in January, 1936, recordings of the water levels in all drill casings of the preliminary borings were made every day or two, and superimposed time graphs and river levels were plotted. Two major flood crests occurred in January, 1936, and one in February. Several of the wells very near the river followed the river fluctuations closely. The others rose very slowly with the repeated floods and were grouped within about 2 ft of El. 558 by the end of February, with little change through March, April, May, and June.

The original natural condition on the south side of the river in June, 1936, before anything had been done to change it, is shown in Fig. 45. The effect of pumping in the cutoff trench in September, 1937, with the cutoff partly completed and with the river partly constricted by the second-stage cofferdam, is also shown in Fig. 45. Fig. 46 demonstrates the remarkably slow response of the ground-water table to the filling of the pool in January, 1939 (if, indeed, the apparent response was not due to heavy rains, as suggested by well No. 25).

In other words, throughout June, 1936 (see Figs. 45 and 46), all wells were steady at approximately 11 ft above the river, which had been steady at El. 547, the extreme low water stage being 544.5. This high and flat water table was in contrast to the north flood plain and indicated little drainage downstream or toward the river.

In August, 1936, pumping in the first trench along the sheet pile cutoff began, and most of the wells were immediately affected, some drawing down as low as El. 520, or 27 ft below the river, which still held practically steady at El. 547. No direct inflow or very rapid infiltration from the river was indicated.

With certain trenches being backfilled and others pumped for many months, the behavior of the wells can only be interpreted by detailed study of all data. In general, until the first winter rise, the river held steady between El. 547 and

El. 548. A number of additional job-made, perforated, well points were driven to rock upstream and downstream from the dam. Some of these went dry during the pumping, indicating an area of free water table somewhere down in the seams and caverns leading to the pumps. At times some wells immediately upstream and downstream from the completed cutoff registered approximately El. 522, which elevation is seen on the placard in the seam shown in Fig. 43. The deepest pumping recorded was in the bottom of the largest cave where the water was drawn down to El. 507.5 at one time, which can be judged from the elevation "514" painted on the rock in Fig. 47. An interesting incident occurred during the final closure of this cave and seam. One of the 36-in. drill cores eventually brought to the surface the piece of stone on which the elevation had been painted. The two adjacent views are of the same cave, with a row of timber posts supporting the roof between. The three large dark voids are the openings of branch caverns, and the black slot near the bottom marks the location of a former shale seam. The arrows indicate the direction of flow of the entering water. El. 507.5 corresponds with the bottom of the general shale seam from which the shale here was missing for some distance, and which seam, being only partly filled with sand and gravel, was bringing water copiously from several directions. This was in November, 1936.

Two wells against the south abutment cliff rose promptly and decidedly at each rain and, after the filling of the pool, well No. 25 was found to register as low as, or lower than, any of the wells on the flood plain during dry periods. Two other wells nearest the river bank consistently held at or about the original El. 548 (see ground-water contour maps, Fig. 45). Occasionally, these wells, and all others, were tested for activity by bailing out or pouring a little water into them. They were situated in an area where comparatively tight clay and silt were known to exist along the south bank of the river.

The behavior of these two holes, the original test pits near the river bank, earth auger holes, the fact that all water flowing into the trenches was clear and cold, and the shape and extent of the pumping drawdown funnels in the water table appeared to indicate that the flood plain was largely cut off from the river and acted as a large filter bed drawing its ground-water supply chiefly from precipitation, from upstream, and from the landward. Later data appear to support this conclusion. As stated, it was known that "chimneys" existed in the rock, and sinkholes existed near the south bluff, so that at times of river overflow the flood plain and seams would be filled quickly. Although two sinkholes later appeared in the borrow pit about 700 ft below the dam, present evidence shows that ground water and such leakage as exists do not drain readily either in a riverward or downstream direction. It may even drain partly southward and into the rock bluff itself, to some lower and unknown horizon (see shape of contours on map of March 26, 1939, shown in Fig. 45).

The ground-water time graphs for the pumping in one of the trenches in September, 1937 (Fig. 45), is one of the few instances suitable for illustration, since the remainder of the graphs are complicated by pumping in adjacent trenches. The accompanying ground-water contour map indicates the extensive drawdown funnel due to pumping, which extends to the south bluff, but is bounded riverward by the known region of tight ground near the river. At

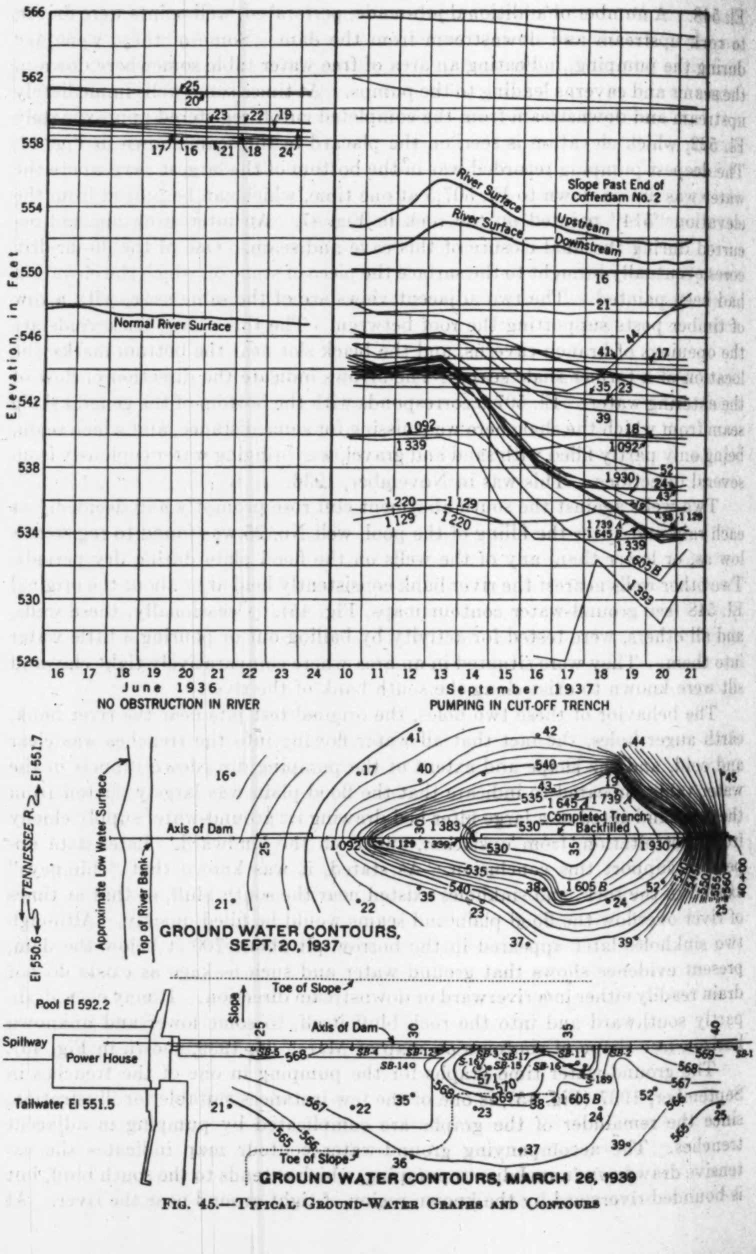

FIG. 45.—TYPICAL GROUND-WATER GRAPHS AND CONTOURS

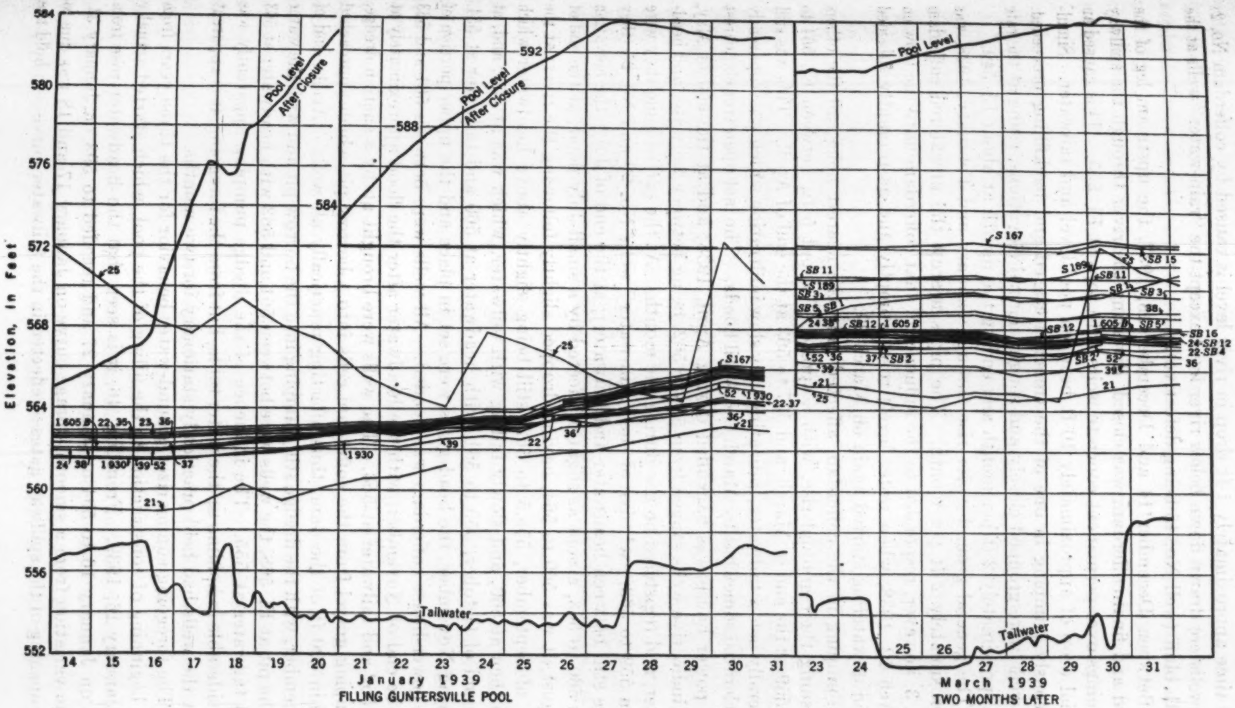

FIG. 46.—TYPICAL GROUND-WATER GRAPHS; JANUARY AND MARCH, 1939

this time approximately 1-ft drop in river level is caused by cofferdam No. 2. All wells are drawn down below river level except the "rain-water" wells at the south bluff (well No. 45 being defective).

Between December 14 and December 18, 1937, the upstream leg of the third and final cofferdam was closed, diverting the river through the spillway openings over permanent concrete weirs with tops at El. 555. This caused an initial drop of approximately 10 ft between pool level and tailwater. Simultaneously pumping in one of the trenches ceased and backfilling proceeded. All wells were grouped closely and sought normal elevations, rising at the rate of approximately 2 ft per month and ending the month at about El. 549.

The general ground-water rise through January and February, 1938, was approximately 2 ft per month. The pipes nearest the unwatered cofferdam No. 3, however, responded to the influence of that cofferdam until the blow on March 12, 1938, when water levels rose promptly and as promptly subsided when unwatering started again on March 29.

Pumping in the cofferdam affected only the nearest holes, all the others pursuing their gradual rise, with a spread of about 5 ft, between El. 554 to El. 559 at the end of March, and 557 to 563 at the end of April. This was still generally below tailwater, indicating very slow infiltration which was apparently accelerated somewhat by March and April floods. The last open trench nearest the power house was backfilled during April, 1938; and at the end of May, the indicative wells ranged from 558 to 562, being between tailwater and headwater, and registered no rise during the month. At the end of June they were from 559 to 563, exclusive of the "rain-water" wells No. 20 and No. 25 (they were still between headwater and tailwater); at the end of July, the elevation was 560 to 563, except as slightly affected by a small July flood; at the end of August, it was 560 to 564, having dropped slightly following the river; at the end of September, 558.5 to 562, still being slightly above headwater which was then at 560, and slowly falling with tailwater, which was at 554; and, at the end of October, 559 to 561, with headwater at 560 and tailwater at 554. During November, the head gates were set in place and the upper portion of the power-house cofferdam was flooded. All wells were between 561 and 563 at the end of November, with the headwater after the flood approximately at El. 562 and tailwater at 553. The wells were brought up by a suction dredge, pumping gravel from the cofferdam cells into a borrow pit which approached within 400 ft of the dam, thus affecting practically all wells. At the end of December, with the dredge still pumping into the borrow pit and with the water in the pit at El. 568, the wells were between 561 and 563 with headwater at 563 and tailwater at 555. The influence of the dredge pumping apparently was considerable in holding up the water table, but from the foregoing, it is apparent that the wells had held practically stationary for two months.

The foregoing summarizes ground-water behavior for the three years from the beginning of construction to the filling of the pool, which started actually on January 16, 1939. From Fig. 46, it is seen that the headwater rose from 567 on January 16 to 594 on January 27 and receded to 593 on January 31. (The variation from a smooth storage curve on January 17 and 18 was due to the opening of the spillway gates—reflected in the tailwater curve—to hold the water until certain obstructions in the lower reservoir had been removed.) The wells rose from a close group average of 562 to 565 during this period, accelerated considerably by frequent rains and particularly by a precipitation of 1.42 in. on January 29. (The occurrence and relative quantity of precipitation can be seen by behavior of well No. 25.) In early February the pool remained approximately at 594, but rising tailwater and rains (such as 2.83 in. on February 2, 1.37 in. on February 6, 1.89 in. on February 10, 0.99 in. on February 14, and 0.75 in. on February 19) brought the wells up in their risers to from 573 to 577 on February 19, with tailwater overflowing the flood plain at 574. Then the tailwater subsided rapidly, and the wells subsided about one half as fast to a close average of 568 on March 1. Although the water table fell approximately 7 ft from February 19 to March 1, that it was free from headwater influence is not fully conclusive since headwater also fell 3 ft during the same period. Further rains in early March, however, brought both tailwater and water table up approximately 2 ft by March 11, after which, with the pool held steady by gate manipulation, except for rising and falling 2 ft over a three-day period, the wells all dropped an average of 3.5 ft to approximately El. 568, with a tailwater drop of 5 ft. The wells held steady until (see Fig. 46), on March 24, the pool was ordered raised from 591 to 594, which it reached on March 30. Although there were rains totaling 2.03 in. during this period, the ground-water table held steady at an average elevation of 568.

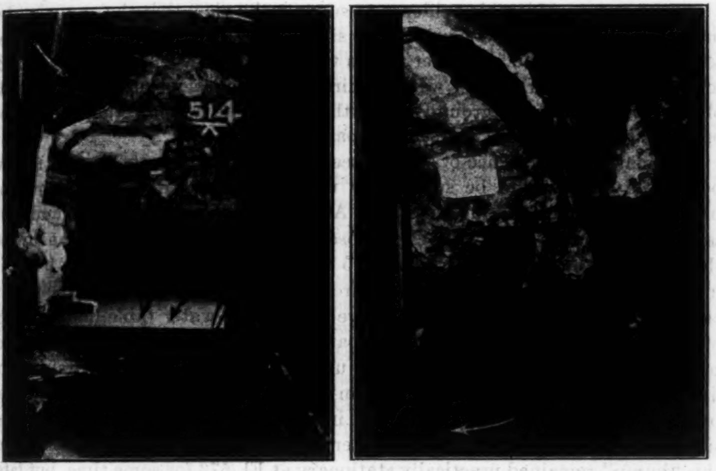

Fig. 47.—Caves in Which Deepest Pumping Was Recorded

Until this time (March 31, 1939) all available data indicated strongly that the observation wells were not influenced by ordinary pool fluctuations. However, the general water table had risen slowly from the average El. 562 to El. 568 (approximately) subsequent to filling the pool, the final lift of the pool being 30 ft, from El. 564 to El. 594. No springs or seepage had shown on the surface near the dam; nor did any appear until, in August, 1939, at several points, approximately 800 or 900 ft downstream in the bottom of a borrow pit, slightly above the level of Wheeler Pool, a seepage flow of approximately 50 gal per min appeared. Chemical analysis of this seepage indicated that it is probably not from the reservoir, or at least to any great extent. Except for this instance, no evidence of springs or seepage came to the surface.

There was considerable evidence to support the view that the ground water moved downstream very slowly; and, since there were no springs or surface seepage except for the 50 gal per min in the borrow pit, seepage past the cutoff must be nominal. However, without direct determinations of direction or rate of the movement of the ground water, there could be no more than considered opinion as to the quantity of seepage past the cutoff wall.

Ordinarily, the absence of surface seepage, or springs at some point not far below the dam, would be sufficiently reassuring. However, there was one phenomenon suggesting further study: After the pool had been filled, and after several subsequent rains, one early observation well (No. 167) began to rise slowly above the others, after about 1.5 months. until by the last of March, in interpolating ground-water contours from the rather widely spaced wells, it caused a "bump" about 4 ft high above the general water table under the toe of the dam. This was corroborated partly by certain adjacent wells. Since practically no water was being picked up by the longitudinal toe drain of the dam, this was interpreted as an artesian effect somewhat similar to the flowing pipe downstream from the north dike, indicating pressure in rock seams, possibly from, or augmented by, deep underseepage from the pool. The height of this "bump" remained practically stationary at El. 573 for some time, but later rose again slowly. Meanwhile, additional wells had been driven as a check and to aid in interpolating contours, as several original wells had been lost during construction and by silt-cement grouting. The new pipes corroborated the "bump" and defined it more clearly, until well SB-22 (see Fig. 48) was driven in the lower slope of the fill and brought in a flow of approximately 5 gal per min at the casing top which was at approximately El. 574. An extension on this casing registered a static elevation of 577.7 on October 3, 1939, raising the top of the "bump" considerably and, with the other wells, moving it farther downstream.

At the time, it was thought that well SB-22 had been driven very close to a "chimney" in the rock, from which seepage water had been rising and diffusing in all directions. Flowing water had been encountered in sand and gravel at about 19 ft below the bottom of the rolled fill as the pipe was driven. It continued after the pipe was driven to rock, through the perforated lower 5 ft, running clear, with no sand, after driving ceased.

A rough attempt was made (with no positive or interpretable result) to determine the direction and rate of flow by means of electrodes in several available wells and a charge of sal-ammoniac in well SB-22. A fan-shaped set of wells, Nos. SB-27, SB-28, SB-29, and SB-30, were then driven to rock at approximately El. 533.7 downstream of the toe of the dam opposite the "bump," for measurements by means of the Slichter apparatus[4] previously constructed and used in 1935 at Pickwick Landing Dam. The Slichter apparatus, at once, gave a rate of flow of 7 ft per hr in a downstream direction. The pervious material here was a layer of gravel 3 in. to 4 in. thick lying on top of rock. From approximate calculations, this would indicate a seepage of approximately 8.75 gal per min per 100 ft of periphery of "bump," which for the apparent length of active periphery would result in the rather nominal total seepage of approximately 53 gal per min.

Slichter tests, checked with an electrical "megger," at a point near the south end of the fill, showed only a slight drop in resistance but nothing interpretable. At one installation on the north flood plain, near the toe of the fill, approximately 100 ft north of the lock, nominal movement was indicated. All installations had been selected, from the ground-water contours, as being most likely to register positive water movement.

It could then be concluded reasonably that probably the only place where there was movement of importance was in the vicinity of the "bump" in the south flood plain. This seemed logical since the "bump" is immediately downstream from the large cave which was found and blocked with a concrete wall under the steel-pile cutoff wall and the general area where the most difficulty was experienced in correcting various seams and caverns. There were arms of these caverns pointing in a general downstream direction.

It had been intended next to drill some of the Slichter holes down into the rock, expecting to strike some of the seams and to determine whether there was measurable movement of water therein. It was not known whether this had ever before been attempted or accomplished. When drilling began in rock, hole No. 28 (approximately 75 ft west of well SB–22) struck a 1-in. seam 12 ft below the top of a rock which produced a flow of 20 gal per min at the casing head, bubbling up 2 or 3 in., with a little sand at first. After a rain, and possible self-cleaning, the flow increased and remained steady at 30 gal per min. Almost instantly after this flow started, the water level in SB–22 and several other pipes forming the "hump" began to fall at a rapid rate (0.8 ft in the first 3 hr). Quite soon the rate decreased and in about 48 hr had become stationary, with a maximum permanent lowering of the previous "hump" of approximately 1.6 ft. It was realized that, by use of "packers," water from rock seams could be controlled if desired; and that the "hump" or pressure zone under the toe of the earth fill could probably (if desired) be reduced further. Therefore, holes SB–27, SB–29, and SB–30 were drilled into the rock. No seams or water was encountered at comparable levels in these holes, and it was decided to try between hole SB–28 and SB–22 near the foot of the dam. All subsequent pipes were not perforated and were seated in the rock before core drilling began. The purpose of thus seating the pipe was to prevent unknown quantities of water, if developed, from escaping without control into pervious gravel, it being desired not only to measure but to control all water liberated from the underlying seams.

Hole SB–46, approximately 46 ft west or downstream from SB–22, promptly brought up 25 gal per min from 12-in. and 24-in. seams, 2.6 ft and 6.9 ft, respec-

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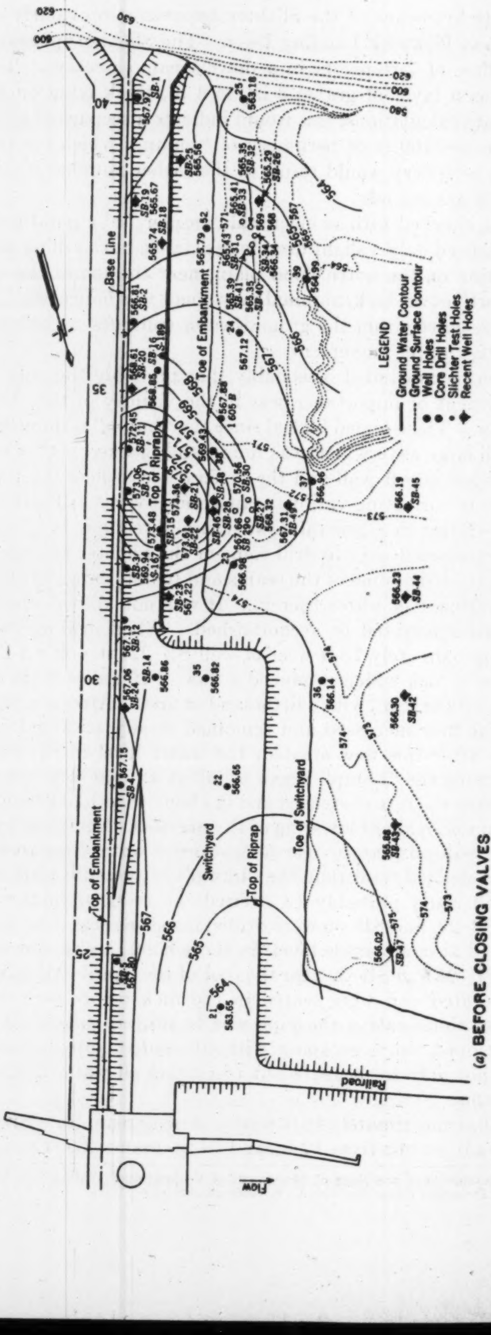
[4] "Field Measurements of the Rate of Movement of Underground Waters," by Charles S. Slichter, *Water Supply and Irrigation Paper No. 140*, U. S. Geological Survey, 1905.


LEGEND
Ground Water Contour
Ground Surface Contour
Well Holes
Core Drill Holes
Slichter Test Holes
Recent Well Holes
Base Line
Top of Embankment
Toe of Embankment
Top of Riprap
Toe of Switchyard
Switchyard
Railroad
Flow
25
30
35
40
564
565
566
567
571
572
573
574
575
580
600
620
630
SB-1
SB-2
SB-3
SB-4
SB-12
SB-14
SB-15
SB-16
SB-18
SB-19
SB-20
SB-21
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SB-45
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SB-47
SB-48
SB-49
563.65
563.18
564.99
565.39
565.56
565.67
565.74
565.79
565.88
566.02
566.14
566.19
566.23
566.30
566.36
566.65
566.81
566.82
566.86
566.98
567.06
567.12
567.13
567.15
567.22
567.31
568.32
568.51
568.85
569.43
570.56
572.45
573.06
573.36
573.48
573.81
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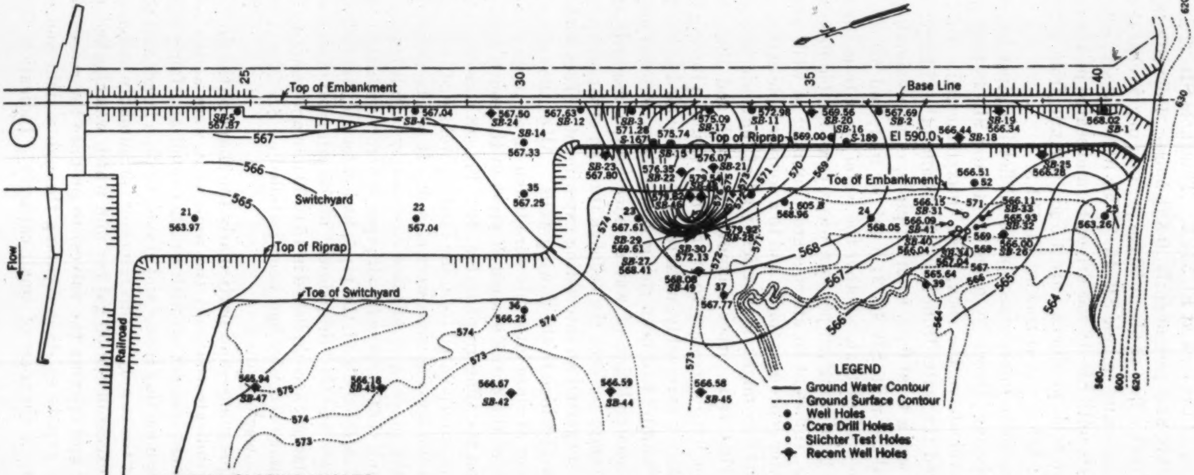
52
189

(a) BEFORE CLOSING VALVES


Top of Embankment
Base Line
Top of Riprap
Toe of Embankment
El 590.0
Switchyard
Top of Riprap
Toe of Switchyard
Railroad
Flow
25
30
35
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567
566
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569
570
571
572
573
574
575
564
580
600
620
SB-5
567.87
SB-4
567.04
SB-24
567.50
SB-12
567.63
SB-14
567.33
SB-3
571.28
S-167
575.74
SB-17
575.09
SB-11
572.98
SB-20
569.56
SB-16
569.00
S-189
SB-2
567.69
SB-18
566.44
SB-19
566.34
SB-1
568.02
SB-25
566.28
SB-23
567.80
SB-15
SB-22
576.35
SB-21
576.07
579.54
579.82
SB-46
SB-29
569.61
SB-27
568.41
SB-30
572.13
579.93
SB-28
SB-49
568.09
21
563.97
22
567.04
23
567.61
35
567.25
36
566.25
37
567.77
24
568.05
1 605
568.96
52
566.51
SB-31
566.15
SB-41
566.09
SB-40
566.04
SB-33
566.11
SB-32
565.93
SB-26
566.00
567.04
39
565.64
25
563.26
SB-47
565.94
SB-43
566.18
SB-42
566.67
SB-44
566.59
SB-45
566.58
LEGEND
Ground Water Contour
Ground Surface Contour
Well Holes
Core Drill Holes
Slichter Test Holes
Recent Well Holes

(b) AFTER CLOSING VALVES

FIG. 48.—WELL-HOLE WATER ELEVATION READINGS BEFORE AND AFTER CLOSING OF VALVES IN FLOWING WELLS SB-28, SB-46, AND SB-48

tively, below the top of the rock at El. 534.3. Thus hole SB–22 and the bump were lowered, quickly and permanently. 0 8 ft more. Hole SB–48, only 12 ft south of SB–46, struck a flow of about 30 gal per min in gravel at the same level as the water-bearing seams in hole SB–46. This new flow also robbed most of the flow from SB–46, until SB–48 was driven to, and seated in, the rock. Drilling SB–48 into the rock produced only 4.5 gal per min from a 2-in. seam at approximately 1 ft below top of rock at El. 521.3.

By now the complete interconnection of all flowing pipes had been established as well as the fact shown by holes SB–46 and SB–48 that the seams in the rock, and gravel above the rock, were interconnected. Drilling in hole SB–46 had muddied the water from hole SB–28 and other like phenomena.

By now there was a total flow, from all pipes, of approximately 56 gal per min (SB–28, 30 gal per min; SB–46, 21.5 gal per min; and SB–48, 4.5 gal per min). It was also evident that, whether the water came from the reservoir or other sources, the total of 56 gal per min was a considerable proportion of the supply to the area, sufficient to affect a great number of wells and lower the "bump" on the water contours rapidly and to a considerable extent. It seemed reasonable that this flow might be practically all of the underseepage (possibly augmented somewhat from other unknown sources) and that the "bump" might represent the head necessary to overcome friction in forcing the water to and out of the wells. Conceivably, until relief was provided by the first flowing well, SB–28, the "bump" had been rising very slowly centered near SB–28 as the apex, due to accumulation of water (since escape channels appeared restricted), in an attempt to force the seepage through other escape channels.

The foregoing suggested an interesting experiment. Tees with outlet gate valves and risers were added to the flowing wells. All of the valves were closed at once and readings at close intervals were taken on all pipes constituting the "bump" and most of the entire system of pipes in the south flood plain, with the result shown on the time graph in Fig. 49. Fig. 48 consists of two ground-water contour maps interpolated from the existing wells just before and shortly after the valves in the flowing wells were closed. It will be noted (see Fig. 49) that the level in the pipes most affected rose nearly to full height in approximately one hour and that practically all pipes, wherever situated, were at least slightly affected. After most of the pipes had come substantially to rest, the valves were re-opened and the "drop down" of the principal pipes was even more remarkable, being largely accomplished in 15 to 30 min and resulting in a concave graph. A water stage recorder had been installed on SB–22 so that the exact shape of this curve was obtained.

Roughly, the foregoing—and particularly the shape of the drawdown curve—would seem to indicate that, (1) the wells that were active and sensitive during the experiment connect (*a*) with a rather small and fairly tight reservoir, consisting of interconnecting seams and gravel, or (*b*) a larger and much tighter reservoir; and that the supply to this reservoir (presumably underseepage) cannot be greatly in excess of the 56 gal per min which was checked and then released (by closing and opening the valves) to produce the recorded effects; or, (2) that whatever supply exists is escaping through a tight and rather small conduit or system of channels of small capacity. Indication (2) seems less

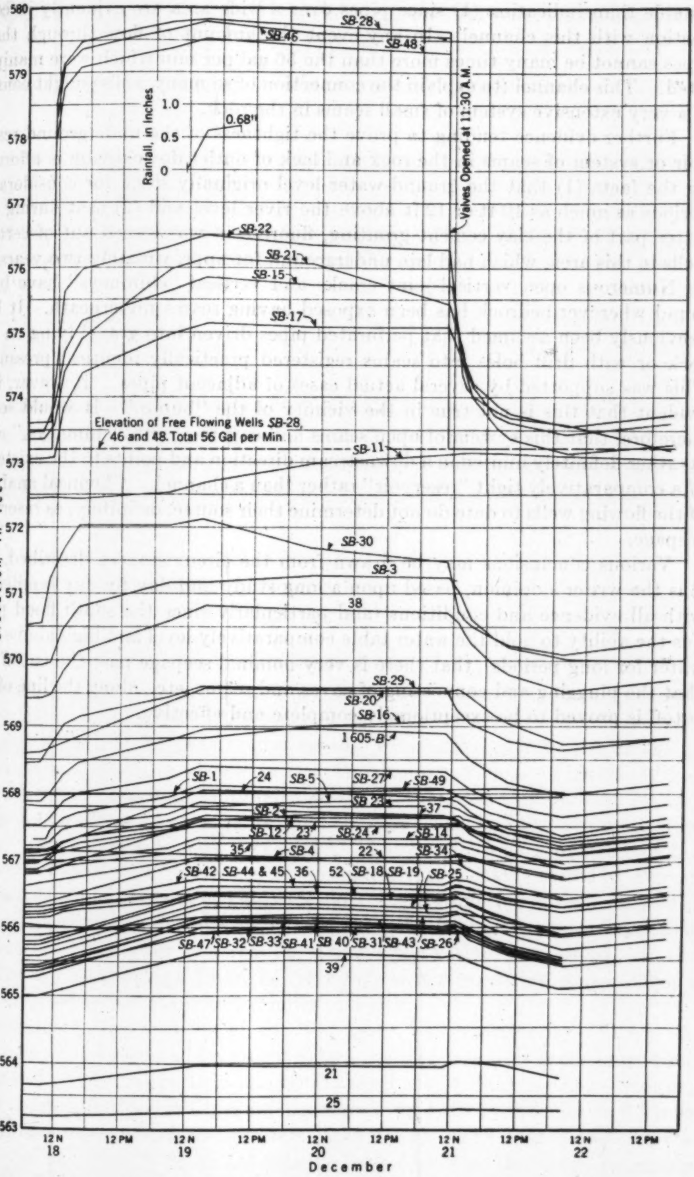

FIG. 49.—TIME GRAPH OF WELLS DURING GROUND-WATER PRESSURE EXPERIMENT

tenable than indication (1) since pipes over a wide area are evidently in connection with this channel. In any event the amount of flow through these pipes cannot be many times more than the 56 gal per min which were manipulated. This channel (to explain the connection of so many wells) might consist of a very extensive system of small seams in the rock.

Further evidence tending to prove the tightness of the underground reservoir or system of seams in the rock and lack of outlet downstream is afforded by the facts (1) that the ground-water level originally stood for considerable periods as much as 10 ft or 12 ft above the river level and (2) that during the latter part of the clay cement grouting, fluorescein was forced out of certain wells in this area, which had lain underground for approximately two years.

Numerous open vertical-joint cracks and vertical "chimneys" have been found wherever bedrock has been exposed having seams underneath. It had previously been assumed that perforated pipes driven into gravel lying on the rock or with drill holes into seams registered practically identical pressures. This was supported by several actual cases of adjacent pipes. However, it is evident that this is not true in the vicinity of the "bump." It would seem, therefore, that this system of open seams and caverns (with "chimneys," etc.) is rather definitely limited in a downstream direction and points to the existence of a comparatively tight "reservoir" rather than a channel. Chemical analyses of the flowing wells to date do not determine their source, definitely, as reservoir seepage.

Various conclusions may be drawn from the circumstances described, but it is the writer's opinion, based upon a long study and day by day familiarity with all evidence and conditions (and particularly since the south flood plain has the ability to hold the water table comparatively level and high above tailwater for long periods), that there is very nominal seepage past the cutoff and that the plugging and connecting of caves and seams, etc., along the line of the cutoff is proved to be exceptionally complete and effective.

FOUNDATION EXPLORATION AND GEOLOGIC STUDIES AT CHICKAMAUGA DAM

By Portland P. Fox,[6] Jun. Am. Soc. C. E.

Synopsis

Engineers and geologists seldom encounter more difficult foundation problems in the construction of a large dam than those which prevailed at Chickamauga Dam. The rock forming the foundation of the dam consists mainly of thin-bedded, pure to argillaceous limestone with thin interstratified beds of bentonite and shale. The rocks are intricately folded and faulted, and are quite cavernous, especially in the zones of structural weakness. All of the foundation problems, including the heavy overburden, are related, directly or indirectly, to the structure and solubility of the rock. By carefully logging, correlating, and plotting all data made available from many hundreds of core-drill holes of both small and large diameters, it was possible to reconstruct the complex foundation conditions, in detail, in advance of actual excavation. This detailed geologic work aided the engineers in determining not only the depth to which excavation was necessary, but also the extent and character of foundation treatment that were necessary.

Introduction

Chickamauga Dam is on the Tennessee River, 471 miles above its mouth and 7 miles upstream from Chattanooga, in Hamilton County, Tennessee. It is the first dam to be constructed on the Tennessee River in the extensive physiographic province known as the Appalachian Valley. This valley lies between the Appalachian Mountains on the east and the Cumberland Plateau on the west and has an average width of 50 miles from central Alabama to New York. It is sometimes spoken of as the "Valley and Ridge Province," or the "Great Valley of East Tennessee." Geologically, it is characterized by a great variety of deformed strata of Paleozoic age. Rocks differing in their resistance to erosion have given rise to reefs, shoals, and pools in the Tennessee River as it meanders southwestwardly across the strike of the various formations. In the vicinity of Chickamauga Dam the river follows a meandering course down a flat-bottomed valley of 0.5 to 1.5 miles in width.

Previous to the creation of the Tennessee Valley Authority (TVA), the U. S. Army Engineers had investigated two possible dam sites in the immediate vicinity of Chickamauga Dam site by diamond-core drilling methods, one just below the mouth of South Chickamauga Creek (mile 468.7) and another 0.9 mile above the present Chickamauga Dam (mile 471.9). The TVA considered the lower site (mile 468.7) to be not feasible for a high dam due to an extremely cavernous foundation, poor abutments, and the great property damage that would be involved in flooding the valleys of North and South Chickamauga creeks. The upper site (mile 471.9) was ruled out in the beginning on the grounds that the bedrock was known to be notoriously cavernous. Later it was deemed wise to explore this upper "Army" site (mile 471.9) in more detail. This later exploration work by the TVA revealed that the top of the rock varies as much as 200 ft and that the bedrock is exceedingly cavernous. In the beginning, the TVA explored a possible site, 0.6 mile downstream (mile 471.3) from the upper "Army" site (mile 471.9), which was on the Chickamauga limestone, but only a few hundred feet downstream from a large overthrust fault. This site, although an improvement over the upper "Army" site, was also quite unsatisfactory due to deep weathering and cavities. The next efforts were to explore two possible sites upstream from the upper "Army" site at miles 472.3 and 474.6. At both of these sites, the top of the rock was found to be very irregular, deeply covered, and cavernous, and therefore unsuitable for a dam foundation. It was then decided to explore a possible site as far downstream as the abutments would allow (mile 471.0) and as far away from the shattered effects of the major thrust faulting as feasible. As expected, this site proved to be the best of the sites explored by either the TVA or the Corps of Engineers and was the site selected. More than 50,000 ft of diamond-drill holes were drilled in the search for this site.

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[6] Geologist, TVA, Spring City, Tenn.

REGIONAL GEOLOGY

The rocks underlying the Appalachian Valley consist of a great thickness of disturbed but unaltered limestones, shales, and sandstones of many formations, ranging in age from Lower Cambrian to Middle Carboniferous.[6] The limestones are of many types, but all are dense, hard, and soluble. The sandstone and shales are likewise of many types.

The once flat-lying Paleozoic strata of limestone, shale, and sandstone, which formerly covered twice the area it now occupies, have been compressed as if placed in a gigantic vise. Today, in crossing the valley, one can see many parallel valleys, trending northeast and southwest, which consist of sedimentary rocks of various kinds, repeatedly faulted so that the blocks lay shingled against one another. All of the major faults are of the reverse or overthrust type and dip from 15° to 60° to the southeast. The prevalent dip of the strata is similarly to the southeast, although local northwest dips occur.

In eastern Tennessee, about 45% of the valley area is underlain by soluble limestone and dolomite, but in the vicinity of Chickamauga Dam the Tennessee River flows entirely on limestone and dolomite for more than 30 miles above Chattanooga. Sinkholes dot the surface, and underground drainage is the rule wherever limestone or dolomite occurs. In the vicinity of Chickamauga Dam the valleys and ridges vary in elevation from 630 ft to more than 1,000 ft above sea level.

As is characteristic of the South, the bedrock about Chickamauga Dam is deeply weathered. In many cases, the uplands are covered with a mantle of residual clay and chert to a thickness of as much as 200 ft.

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[6] "Geology of Alabama," by G. I. Adams, Charles Butts, L. W. Stephenson, and White Cooke, *Special Report No. 14*, 1926.

DESCRIPTION OF THE BEDROCK

Due to much folding and faulting, and the length of the dam, an unusual thickness and variety of limestone and shale strata were encountered in the excavations for the construction of Chickamauga Dam. This group of mainly thin-bedded, dark colored, hard, soluble limestone, and thin zones and beds of shales, has long been known as the "Chickamauga limestone."[7] This type of limestone includes approximately 2,000 ft of the Upper Ordovician system and several similar, but well-known, formations. Much could be written in describing these formations if space permitted, but only the Trenton and upper Lowville formations are described in any detail since they immediately underlie the principal features. Fig. 50 shows the order, areal distribution, and structural relation of the formations to the dam and surrounding area.

The Trenton formation,[8] the upper part of the Chickamauga limestone, forms the major part of the foundation, immediately underlying practically all of the main structures except the south half of the south earth dam. The Trenton is about 175 ft thick, and is composed of two distinct types of limestone. The upper Trenton, or Cannon limestone, formed the surface rock over most of the area around the lock, spillway, and power house, as shown in Fig. 50, and varied in thickness from 0 to 110 ft. This limestone is dark gray to black in color, fine to medium crystalline, and medium bedded. Cavities up to a vertical and horizontal dimension of 25 ft were sometimes present in this formation, practically all of which was removed by excavation under the lock, spillway, and power house. The lower Trenton, or Hermitage limestone, consists of 65 ft of thick-bedded, impure, mottled limestone and several thin beds of bentonite and shale. Eighteen samples of the limestones of the Trenton formation gave an average compressive strength of 16,900 lb per sq in.

Upper Lowville or Tyrone Formation.—This formation immediately underlies the Trenton and is penetrated by most of the deep drill holes under the spillway, power house, and lock. It also forms most of the rock exposed in the left or south abutment. The Lowville, like the Trenton, is composed of two distinct types of limestone. The lower part has been given the name "Carters" and the upper part "Tyrone" in Tennessee. The lower part is very similar to the underlying rocks and is included with them in the description. The Tyrone consists of 35 ft of thin beds of gray, fine-grained limestone; greenish-gray and red, shaly limestone; and several beds of bentonite. This formation has many distinctive characteristics and has served as a valuable horizon marker in recognizing faults.

The other formations mentioned are described briefly with the south half of the south earth dam, the only place where they occur in the foundation at the surface.

Beds of Volcanic Material.—Interstratified at various intervals with the limestone of the upper Lowville (Tyrone) and the lower Trenton (Hermitage) formations are thirteen beds of soft, white, shaly and sandy clay-like material known to be derived from volcanic ash. This material varies in the size of the individual particles from extremely fine to coarse. The fine, clay-like material

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[7] "Chattanooga Folio No. 6," by C. W. Hayes, U. S. Geological Survey, Atlas, 1894.

[8] "Stratigraphy of the Central Basin of Tennessee," by R. S. Bassler, *Tennessee Survey Bulletin 38*, 1932.

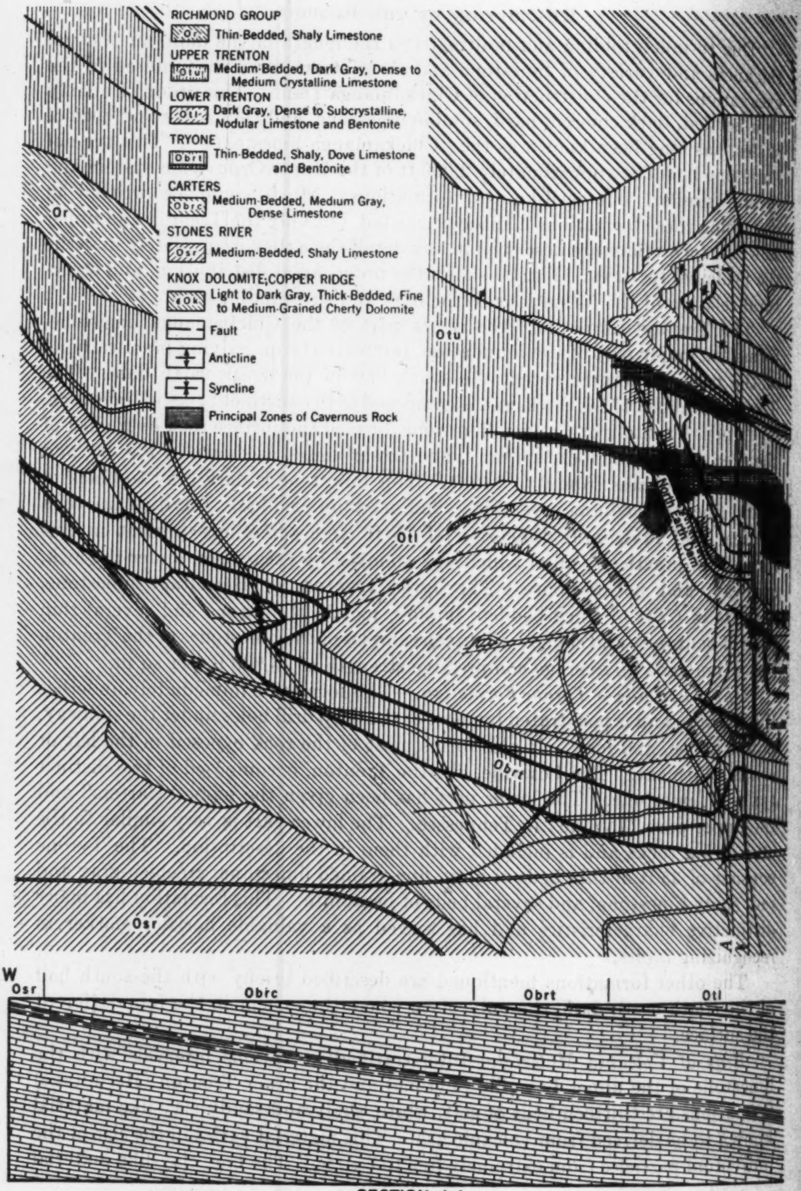
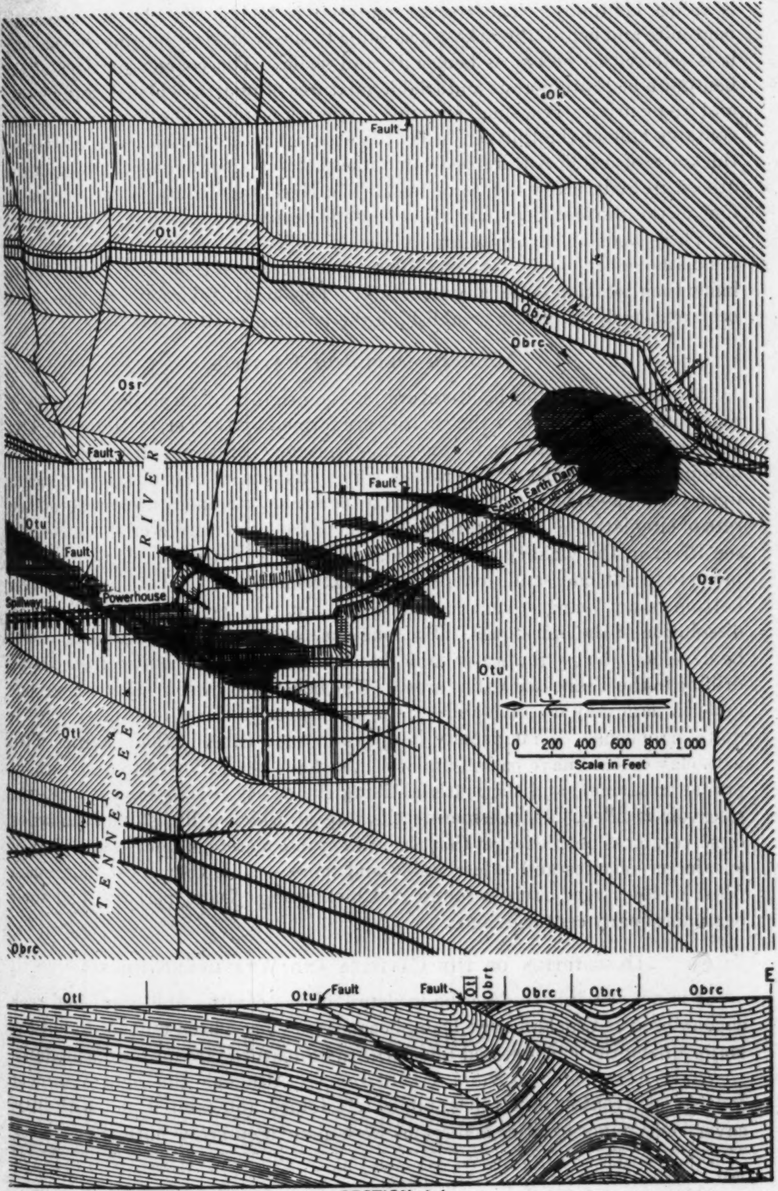

RICHMOND GROUP
Or
Thin-Bedded, Shaly Limestone
UPPER TRENTON
Otu
Medium-Bedded, Dark Gray, Dense to Medium Crystalline Limestone
LOWER TRENTON
Otl
Dark Gray, Dense to Subcrystalline, Nodular Limestone and Bentonite
TRYONE
Obrt
Thin-Bedded, Shaly, Dove Limestone and Bentonite
CARTERS
Obrc
Medium-Bedded, Medium Gray, Dense Limestone
STONES RIVER
Osr
Medium-Bedded, Shaly Limestone
KNOX DOLOMITE;COPPER RIDGE
€Ok
Light to Dark Gray, Thick-Bedded, Fine to Medium-Grained Cherty Dolomite
Fault
Anticline
Syncline
Principal Zones of Cavernous Rock
Or
Otu
Otl
Obrt
Osr
North Earth Dam
A
A
W
Osr
Obrc
Obrt
Otl
SECTION A-A

FIG. 50.—GEOLOGIC MAP OF SITE, SHOWING DIS-


Ok
Fault
Otl
Obrt
Obrc
Osr
Fault
RIVER
Fault
Otu
Fault
Spillway
Powerhouse
South Earth Dam
Osr
Otu
Otl
TENNESSEE
Obrc
0 200 400 600 800 1000
Scale in Feet
Otl
Otu
Fault
Fault
Otl
Obrt
Obrc
Obrt
Obrc
E
SECTION A-A

TRIBUTION OF FORMATIONS AND CAVERNOUS AREAS is called bentonite. The coarser material is technically classed as arkosic bentonite and bentonitic arkose according to the grain size; but for convenience it is referred to herein as bentonite and shale. The coarser portions of the bentonite beds are porous and friable mixtures of quartz, feldspar, and biotite, and frequently carry a small amount of artesian water. The Tyrone formation contains nine of the thirteen beds of bentonite, two of which have an average thickness of 3 ft. A third bed sometimes attains a similar thickness, but the others are only a few inches or less. The Hermitage contains the remaining four of the thirteen beds, but all of these beds are less than 6 in. thick.

These beds of bentonite are extremely difficult to core satisfactorily with the diamond drill, but it was very important, because of their weak character, to know their location under any structure. The 36-in., or larger core holes, have been extremely helpful in establishing the position of these soft beds.

Mode of Origin of Cavities

Much can be found in literature on the origin and formation of caves in limestone. In most cases the cavities are formed by the percolation of slightly acid ground water through some small primary or secondary weakness or opening in the bedrock. The rate at which any limestone is dissolved depends upon the area exposed to the water and the acidity, temperature, pressure, and rate of circulation of the water. It is evident from the character of the Tennessee River in the vicinity of Chickamauga Dam that it is unable to transport more than silt and sand, except during extreme floods. Therefore, the river bottom is not being eroded mechanically, but it is being lowered slowly by dissolution of the limestones. The long-accepted concept that cavities cannot be developed below the water table has been shown to be quite erroneous by drilling in the Tennessee Valley area.[9] The belief held by some that a limestone under a dam may dissolve and become cavernous in a few months or years is quite erroneous. What usually happens in such cases is that cavities filled with clay or sand have passed unnoticed, or have been allowed to pass without proper treatment, and the material filling the cavity is soon eroded when the reservoir is filled. In considering the rate at which a limestone foundation dissolves, most people forget that the cement in concrete is composed mainly of tricalcium silicates and is less stable and more soluble than most limestones. Most of the larger cavities found at Chickamauga Dam, perhaps, had their beginning many centuries ago.

Description of the Cavities and Weathered Rock

It is not possible to describe in detail all the cavities and weathered rock encountered at Chickamauga Dam, because they have been numerous, but the major difficulties encountered under the main features are outlined as follows.

Right Abutment and North Earth Dam.[10]—Although the right abutment consists of Trenton limestones, intricately folded and faulted and with little cover, only minor cavities and seams were discovered below normal pool level, or 682 ft above mean sea level (see Fig. 51). The north earth dam, being 1,365 ft long, stretches across the former North Chickamauga Creek channel and an old flood plain built by the Tennessee River and the creek. This flood-plain material, consisting of gravel, sand, and clay, lay on bedrock or boulders and residual clay at El. 625 and extended as high as 670 ft above mean sea level at a few places. At no place under this entire earth dam did sound bedrock lie immediately below the flood-plain material. Beginning at the east end of the north earth dam, for the first 420 ft along the axis of the dam, high dips, small faults, many small seams and cavities, and a few large cavities, prevailed in the upper 30 ft of the bedrock. In the next 120 ft along the dam axis, the top of the rock was depressed into a U-shaped, sinkhole-like depression to a low elevation of 531, or about 90 ft lower than it was either to the east or to the west. This depression in the rock, in its natural condition, was filled with a yellow, residual, plastic clay and chert. For 300 ft west of this depression, also, the bedrock was severely weathered along joints, small faults, and bedding planes. This depression, and the weathered condition west of it, is due to a zone of many small faults, or a fracture zone. West of this point, to the lock, the bedrock lay practically flat. In this latter zone, solution had occurred along nearly vertical joints to a depth of 10 ft, and some large limestone boulders remained over the bedrock.

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[9] "Deep Solution Cavities in the Tennessee Valley Area," by Berlen C. Moneymaker, *Proceedings for 1937*, Geological Soc. of Am., June, 1938, p. 101.

[10] "Elements of Chickamauga Dam," by Lee G. Warren, M. Am. Soc. C. E., *Engineering News-Record*, October 21, 1937, p. 667.

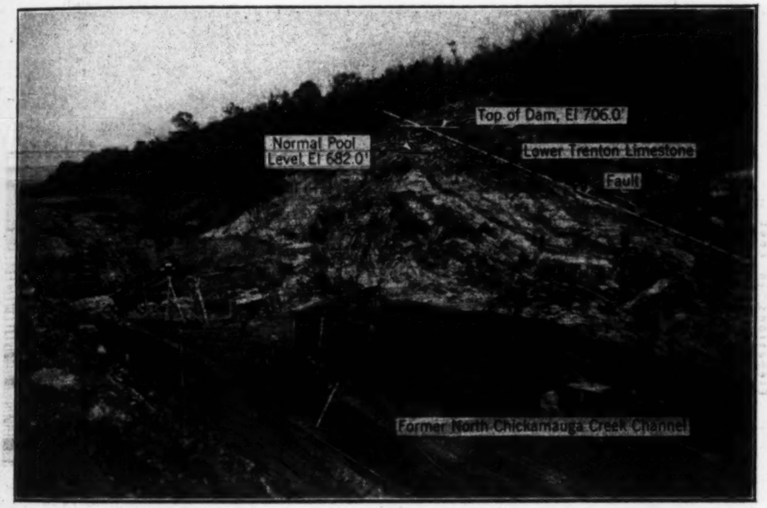
Top of Dam, El 706.0'
Normal Pool Level, El 682.0'
Lower Trenton Limestone
Fault
Former North Chickamauga Creek Channel

Fig. 51.—North Abutment Showing Folded and Faulted Trenton Limestone

The Lock.—A depression in the top of the rock of about the same magnitude as the one encountered under the north earth dam was also encountered under the lock piers.[11] The top of the rock under the lock varied in elevation from 625 to 535 ft. Under the downstream guard walls, several small faults,

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[11] *Proceedings*, Am. Soc. C. E., June, 1939, Fig. 3, p. 956.

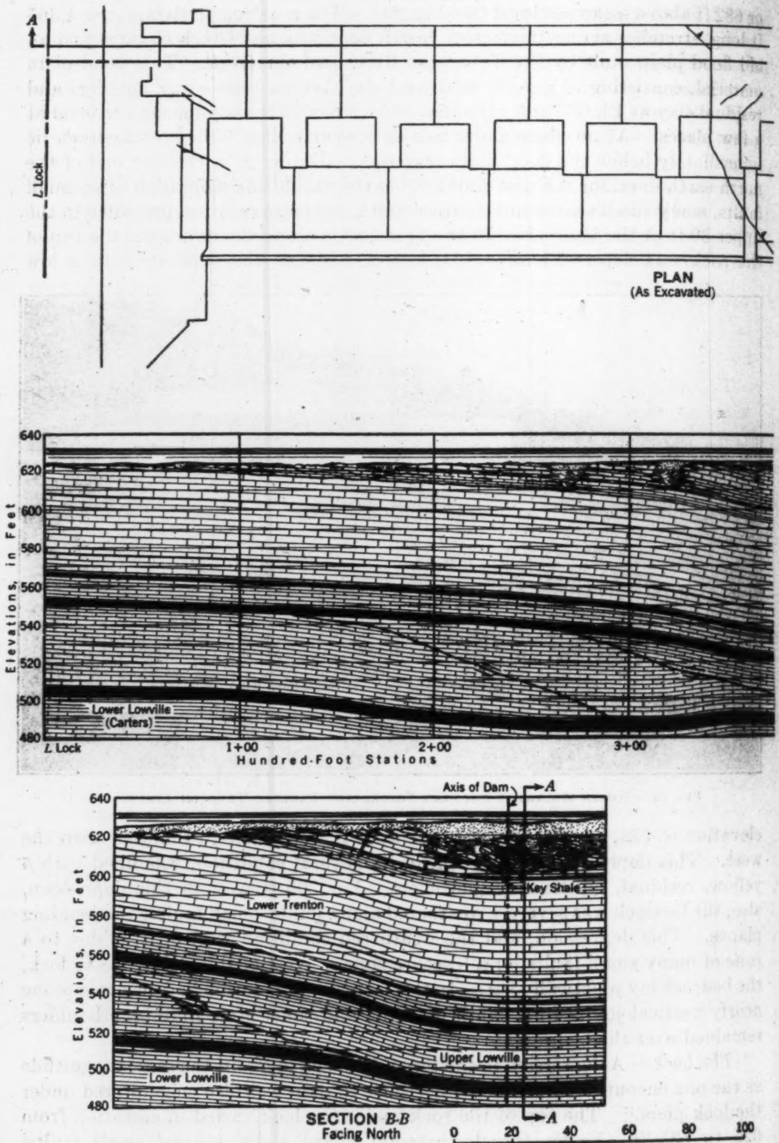
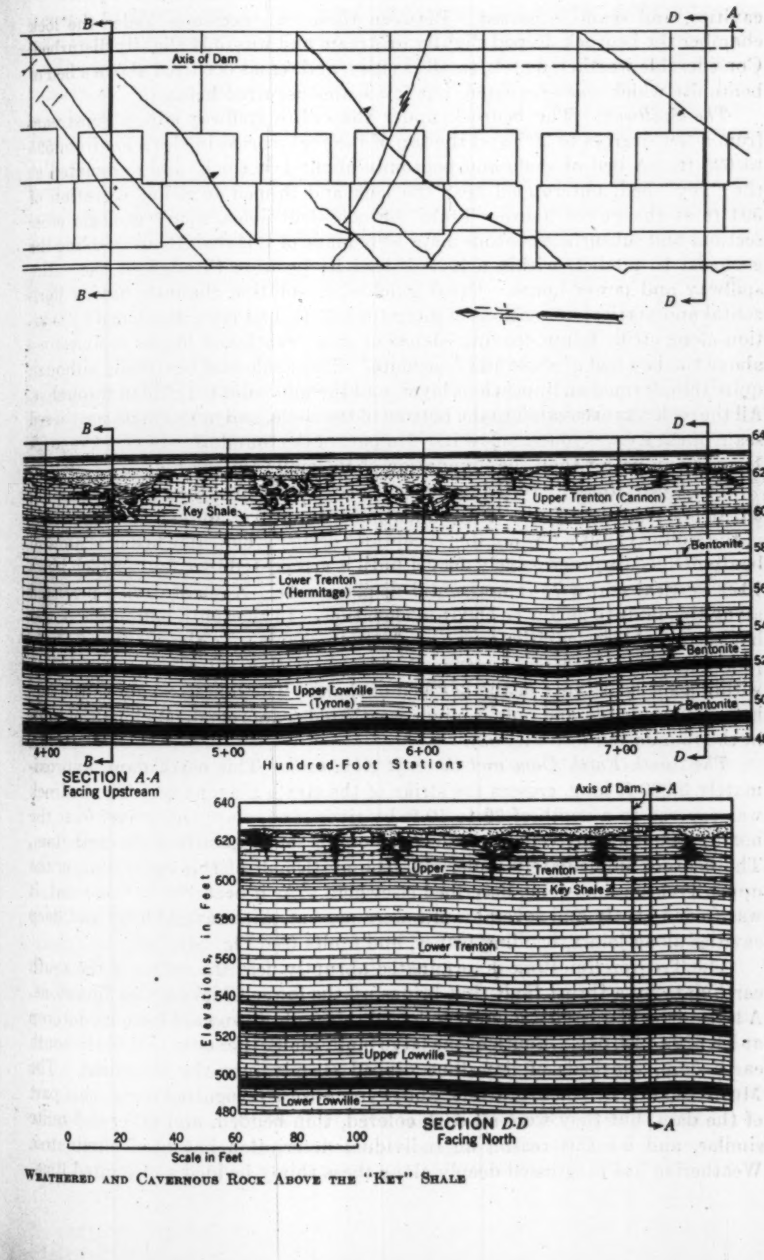

A
℄ Lock
PLAN
(As Excavated)
640
620
600
580
560
540
520
500
480
Elevations, in Feet
Lower Lowville
(Carters)
℄ Lock
1+00
2+00
3+00
Hundred-Foot Stations
Axis of Dam
A
Key Shale
Lower Trenton
Upper Lowville
Lower Lowville
A
SECTION B-B
Facing North
0
20
40
60
80
100
Scale in Feet

FIG. 52.—GEOLOGIC CROSS SECTION UNDER THE SPILLWAY, SHOWING


B
D
A
Axis of Dam
B
D
B
D
640
620
600
580
560
540
520
500
480
Elevations, in Feet
Upper Trenton (Cannon)
Key Shale
Bentonite
Lower Trenton
(Hermitage)
Bentonite
Upper Lowville
(Tyrone)
Bentonite
4+00
5+00
6+00
7+00
B
D
Hundred-Foot Stations
SECTION A-A
Facing Upstream
Axis of Dam
A
Upper
Trenton
Key Shale
Lower Trenton
Upper Lowville
Lower Lowville
A
SECTION D-D
Facing North
0
20
40
60
80
100
Scale in Feet

WEATHERED AND CAVERNOUS ROCK ABOVE THE "KEY" SHALE cavities, and seams occurred. Between these two extremes under the lock chamber the bedrock dipped slightly upstream and was only slightly disturbed. Considerable weathered rock, small cavities, and seams occurred above a bed of bentonite which was excavated, but few seams occurred below it.

The Spillway.—The bedrock under the entire spillway dipped upstream from a few degrees to 20°, and the top of the rock varied in elevation from 595 to 625 ft. A bed of shale and bentonite about 1 ft thick, and designated as the "key" bed, outcropped near the lock and dipped to a low elevation of 594 ft at the power house. From the core-drill holes, many geologic cross sections and subsurface contour maps were made of this shale, thus enabling the geologist to predict, within a foot or less, its position throughout the entire spillway and power house. Great gulches, or solution channels having horizontal and vertical dimensions of more than 25 ft, had been developed by solution along strike joints, leaving islands or large weathered blocks of limestone above the key bed of shale and bentonite. This shale and bentonite, although quite thin, formed an impervious layer, and the joints did not extend through it. All the rock was excavated to the bottom of the shale, and not a single weathered seam or cavity was found below it. No part of the foundation of Chickamauga Dam is any better than the foundation of the spillway (see Fig. 52).

The Power House.—In the power-house area the top of the rock varied from 582 to 625 ft in elevation. The foundation of the power house is similar to that of the spillway, but a few notable exceptions occur. The same bed of shale and bentonite occurs progressively deeper until it is repeated by a fault, which has a 10-ft vertical and a 75-ft horizontal displacement. A small cavity persisted along this fault to as low as 560 ft in elevation. In the south half of the power house several large cavities occurred above this fault. In the north end of the power house a mass of limestone boulders and clay extended downward to within a short distance of the key bed of shale. The limestone above the key bed was excavated and only a minor weathered seam remained along the fault in the foundation (see Fig. 53).

The South Earth Dam and the Left Abutment.—This earth dam, approximately 3,000 ft long, crosses the strike of the strata diagonally. The bedrock was covered to a depth of 30 to 40 ft by river sand, clay, and gravel over the north 2,000 ft and by yellow residual clay on the south 1,000 ft of the earth dam. The bedrock immediately underlying the north half of this earth dam is the upper Trenton limestone, described elsewhere. As it occurred at this point, it was folded gently and several faults were present, with several large and deep cavities along joints, bedding planes, and faults (see Fig. 54).

The Trenton limestone is terminated abruptly near the center of the south earth dam by a thrust fault that brings up the lower Chickamauga limestone. A large, irregular cavity roughly followed this fault downward from its outcrop at El. 620 to El. 545. South of the large fault, under the south half of the south earth dam, the beds of limestone dipped steeply into the abutment. The Murfreesboro, Lebanon, Carters, and Tyrone were recognized under this part of the dam, but they were all dark colored, thin bedded, and otherwise quite similar, and for this reason an individual description has been eliminated. Weathering has progressed deeply along these thinly bedded and jointed limestones. At several places clay seams and large cavities were in abundance to a depth of 40 ft below the top of the rock, which required deep excavation.

The bedrock in the left abutment, covered by 40 to 60 ft of residual clay, consists, at the bottom, of a bluish-gray limestone and, at the top, of a series of red and green shaly limestone with several thick beds of bentonite interstratified. This latter series is known as the upper Lowville or Tyrone, described elsewhere. Although the top of the rock in this abutment is slightly below normal pool level, and some shallow filled cavities occur in the bedrock, it is considered satisfactory due to the great thickness of residual clay overlying the rock and the structural advantages that the beds of bentonite afford.

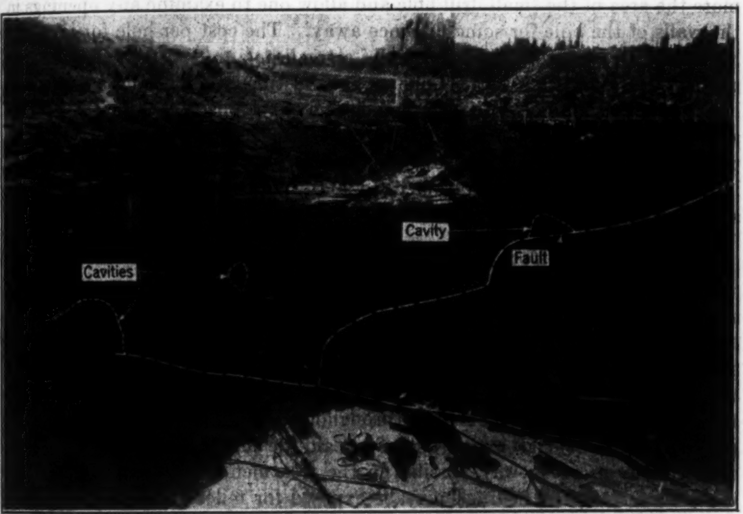


Fig. 53.—Power-House Excavation, Showing Fault and Cavities in Upper Trenton Limestone

Type of Core Drills and Methods Used for Exploration

Practically all small core-drill holes, for purely exploration purposes, have been driven with special equipment operated by hydraulic feed, using a swivel type of double-tube core barrel. Many shot-drill holes ranging in size from 3 to 8 in. have been drilled by several models of shot drills for foundation treatment purposes. Also, a considerable number of large holes, ranging in diameter from 32 to 52 in., have been drilled by equipment described elsewhere.[13]

In exploring a limestone foundation where many soft, weathered seams occur and where many beds of bentonite and shale are interstratified with the limestone, it is imperative that the core recovery be kept as nearly 100% as possible. If the drill water returns, much can be learned by keeping notes on its color, and by comparing with the core losses; but at Chickamauga Dam the drill water was almost invariably lost at the top of the rock or in the first cavity. For exploration purposes under such conditions, the engineers found that a diamond drill using an hydraulic feed, a double-core barrel, and a special bit giving a 2⅛-in. core is by far superior in core recovery to any other type of small core drill. Smaller bits have been used, but the core recovery is much less satisfactory. The small shot drills proved highly unsatisfactory for exploration purposes in coring soft and weathered rock.

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[13] *Proceedings*, Am. Soc. C. E., June, 1939, p. 955.

The large shot-drill holes of 32 to 52 in., which permit the geologist to enter and inspect the undisturbed rock, is yet unsurpassed by any exploratory drilling method.[13] Where the conditions of the bedrock are as variable as those at Chickamauga Dam, the core from a small drill hole may not represent, even partly, the area immediately around it. The large holes cover 100 times or more the area of the small drill hole and allow one to examine any openings in the walls of the hole for some distance away. The cost per hole for the large hole is usually several times the cost of a small hole, but also the information gained and value of the larger hole are several times that gained from a small hole. A great number of these large holes have been used both for exploration and foundation treatment. It would have been difficult to explore this site thoroughly without their use.[14] On January 1, 1939, 4,618 ft, or 99 of these large holes, ranging in depth from 15 to 90 ft, had been drilled during the construction of this dam.

FUNCTIONS OF THE GEOLOGIC STAFF

Without doubt, the greatest value a geologist can render is in the proper selection of a dam site. Nevertheless, if the site is located on a much folded and faulted limestone such as that at Chickamauga Dam,[15] even after considerable drilling has been done, many "gaps" always remain that can be filled in best after construction has started. For this reason, a two-man geologic staff was maintained at Chickamauga throughout the construction period. The principal aims of the geologic staff were to aid the engineering and construction departments in every way possible in the treatment of the foundation. At many dam sites, the bedrock may be exposed for miles upstream and downstream, and a brief investigation by a consultant geologist is all that is required, but at Chickamauga Dam outcrops of the bedrock in the abutments were of little value in developing the details of the foundation. Most of the bedrock was covered on the flood plains by a 30-ft to 40-ft layer of recent river gravel, sand, and clay, and on most of the abutments by residual clay or older deposits of river material.

For these reasons, it was necessary to depend almost entirely for geologic information on small-diameter shot-drill and diamond-drill cores and on 32-in. to 54-in. calyx drill holes. The aim was to log and record for future use an accurate description of every foot of core recovered and of every 36-in. hole drilled. This may seem to be a small task, and at many dam sites it would be, but at Chickamauga Dam it has been a large part of the work.

It was found necessary to have a very detailed geologic description of the core from every hole in order to make accurate geologic cross sections for

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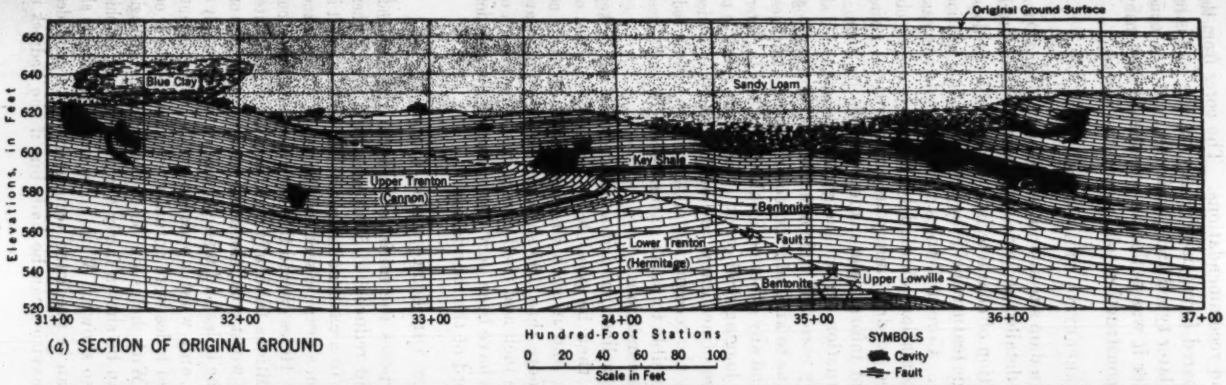
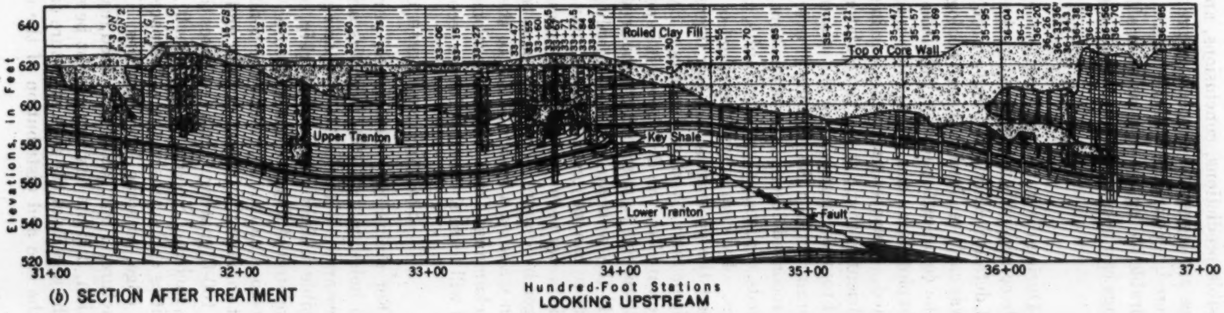
[13] "Core Drilling for Visual Examination of Foundation Material," by B. M. Jones, M. Am. Soc. C. E., Third World Power Conference and Second Conference on Large Dams (Section A on Dams), September 1–6, 1936; also *Proceedings*, Am. Soc. C. E., June, 1939, p. 953.

[14] "Large-Diameter Core Drill for Geologic Exploration," by Berlen C. Moneymaker and Portland P. Fox, A.I.M.E. *Tech. Pub. 1000*, 1938.

[15] "Bad Rock Limits TVA Dam Location," *Engineering News-Record*, October 21, 1937, p. 665.


Original Ground Surface
Blue Clay
Sandy Loam
Key Shale
Upper Trenton
(Cannon)
Bentonite
Fault
Lower Trenton
(Hermitage)
Bentonite
Upper Lowville
Elevations, in Feet
660
640
620
600
580
560
540
520
31+00
32+00
33+00
34+00
35+00
36+00
37+00
Hundred-Foot Stations
0
20
40
60
80
100
Scale in Feet
SYMBOLS
Cavity
Fault

(*a*) SECTION OF ORIGINAL GROUND


F-3 GN
F-3 GN 2
F-7 G
F-11 G
F-15 GS
32+12
32+25
32+60
32+75
33+06
33+15
33+27
33+47
33+55
33+60
33+66.5
33+67
33+71
33+77.5
33+84
33+88.7
Rolled Clay Fill
34+30
34+55
34+70
34+85
35+11
35+21
Top of Core Wall
35+47
35+57
35+69
35+95
36+04
36+12
36+20
36+26 A
36+33'36"
36+34.5
36+38
36+48
36+56
36+70
36+85
Upper Trenton
Key Shale
Lower Trenton
Fault
Elevations, in Feet
640
620
600
580
560
540
520
31+00
32+00
33+00
34+00
35+00
36+00
37+00
Hundred-Foot Stations
LOOKING UPSTREAM

(*b*) SECTION AFTER TREATMENT

FIG. 54.—GEOLOGIC SECTION UNDER SOUTH EARTH DAM, SHOWING FAULTS, FOLDS, AND CAVERNOUS ROCK

sensible predictions, conclusions, and recommendations. The cores from the holes are logged on a specially prepared form, day by day, as the core is recovered. Four copies of the log are later typed and distributed. It was found desirable to log the core as soon after it was pulled as possible because many things can happen to the core on a construction job to spoil the record.

Method of Interpreting the Cores

During the initial exploration for a dam site at and near Chickamauga Dam, little consideration was given to the detailed geologic structures of the bedrock and detailed description and correlation of characteristic beds and formations. It was early realized that such geologic features held the key to the final solution of the complex foundation problems. Core-drill holes are an expensive means of exploration, and all possible information afforded by them should be obtained. Some engineers are inclined to drill numerous cores without giving sufficient geologic study to the latent information made available by them.

From the beginning of the construction period, the geologists assembled all information in a detailed log of every core-drill hole completed. In these logs are recorded descriptions of the core as to kind of rock, color, texture, thickness of beds, dip, seams, cavities, faults, and any other noticeable physical characteristics. By such detailed method of logging drill cores, it has been possible to depict the geology with a fair degree of accuracy, although it may be buried 30 ft or more by river deposits. By the recognition of the characteristic beds penetrated by the drill holes, it is possible to make natural-scale geologic cross sections and thus to picture the true conditions.

If the beds are not too steeply inclined, and if some definite bed or key horizon can be recognized, it may then be possible to connect this bed or horizon between various drill holes, and in such cases oriented cores are no longer necessary. By this means more than 100 such geologic sections have been made on a scale of 20 ft to the inch and used during the construction of Chickamauga Dam. These sections have been very useful during construction and will form a valuable future record of the foundation.

Spacing and Depth of the Exploration Holes

For preliminary exploration purposes the initial spacing of the diamond-drill holes was on 100-ft centers and reduced to 50 ft at a few places when necessary. With the 100-ft spacing in a cavernous limestone foundation, it is possible to show only very roughly, on a graphic section, the top of the rock and a skeleton of the geologic structure. Holes spaced this far apart in a cavernous limestone cannot represent the conditions faithfully. It was discovered later that some of the largest cavities and worst conditions fell between the holes on the 100-ft spacing. In a few cases the holes had missed large cavities by only a few inches. Several of the small, and what appeared to be unimportant, cavities and seams in the wide-spaced holes were actually the small ends, or on the edge, of some large cavities. With holes on 50-ft centers, the geologic structure, if not too complicated, can be shown fairly accurately on a graphic section, but the cavities cannot be so shown. With holes on 25-ft centers all of the details of the rock structure can be predicted accurately and a good idea of the size and distribution of the cavities can be had; but if the bedrock is extremely cavernous, holes spaced on 12.5-ft centers will be of considerable aid in planning the methods of treatment. These closer holes have been drilled on some one definite cutoff line and where they can be later used for grout holes. The advantages of placing all the holes along one line rather than spacing the same number of holes over a large area is that the geology can be traced in detail and the good and poor sections can be projected with a fair degree of accuracy which otherwise would be impossible.

Only a limited number of the diamond-drill holes have been driven on an angle from the vertical or horizontal. Where cavities occur along vertical joints in horizontally bedded limestone, angle holes can be used with greater value than vertical ones; but at Chickamauga Dam, where nearly horizontal beds occurred, cavities were shallow and few needs for angle holes arose.

The depth of the exploration holes was varied between 50 and 200 ft in order to encounter faults, beds of bentonite, and zones of deep weathering. On January 1, 1939, 58,244 ft of diamond and 47,772 ft of small shot-drill core had been drilled and carefully logged by the geologic staff during the construction of the dam.

Geophysical Method of Exploration

Three attempts were made to explore the conditions of and the depth to bedrock by electrical resistivity measurements. Considerable data were obtained by the patented tester, and the great difficulty was not with operation of the tester itself, but in the interpretations of the data obtained. Perhaps the main difficulties in interpretation of the data were due to an overburden of variable character and quite variable conditions in the bedrock which were not indicated by the widely spaced drill holes. This method could be used best before drilling so that no drill casing or water lines would be in the way to affect the resistivity measurements; and if any unusual conditions were noted they could be checked with drill holes at the beginning. By this means it would be possible to investigate the worst conditions of a proposed dam site first and not after many drill holes had been driven.

Source of Information and Acknowledgments

The writer was connected with the geologic staff at Chickamauga Dam from the construction in 1936, and most of the information presented herein has been obtained first-hand from his own observations and with the assistance of Leland F. Grant and John C. Dunlap. The writer is also indebted greatly to many engineers on the engineering and construction staffs at Chickamauga Dam. Most of the preliminary geologic exploration was in the care of Harry N. Eaton, Robert M. Ross, Roy Caldwell, and William Lovelace.

Conclusions

The exploration and geologic studies at Chickamauga Dam clearly revealed the structure, character of the bedrock, and the extent, relation, and depth of the cavities and weathered rock. The main problems at Chickamauga were related to the foundation, and geologic studies were of material aid in overcoming difficulties in the construction of the dam. Fortunately only a few dam foundations are as complicated as this one, but in such cases only painstaking detailed geologic studies can satisfactorily unravel the real nature of the foundation problems.

FOUNDATION TREATMENT AT CHICKAMAUGA DAM

By James B. Hays,[16] M. Am. Soc. C. E.

Synopsis

Chickamauga Dam is on the Tennessee River, 7 miles above the City of Chattanooga, Tenn. The river flows westerly at this point. The project consisted of a lock adjacent to the north, or right, bank; a spillway across the main part of the stream; and a power house on the south, or left, bank, with two sections of earth dam, one on the right flood plain 1,300 ft long, and another on the left plain 2,900 ft long, known as the north and south earth dams, respectively, making a total of 5,900 ft. In plan, the structure is a flat U shape. The axis was curved downstream to follow the best foundation rock available. Fig. 55 shows the general plan of the dam.

Due to the nature of the foundation rock, with its numerous channels, cavities, seams, faults, and folds, the problem of making a watertight cutoff was one of greatest importance. Perhaps at no other dams were more thorough investigations made. Grouting covered a wide range, using different materials and methods, and it is felt that, due to the extensive nature of the work involved, the information and experience obtained on the project will prove of value to others.

The cutoff was grouted, using only cement and water. Other grouting for temporary construction purposes, and to fill cavities under the earth dams, was done with other materials and mixtures.

Introduction

Description of Sequence of Operations.—Construction began in January, 1936, the work in the river being done in three stages. The construction of the lock, at the right bank, was the first stage and was completed, ready for temporary operation, by June, 1937. The lock chamber is 60 ft wide by 360 ft long and provides for a lift of 50 ft. The second stage followed and included the construction of 14.5, of a total of 18, bays of the spillway. Each bay consisted of a 40-ft gate opening and an 8-ft pier. This stage was completed in the winter of 1937–1938. Stage three included the remainder of the spillway and the power house. This was started during the spring of 1938. During the third stage, the river was diverted through the spillway. During the operation of the foregoing program, the earth-dam sections and the diversion channel for North Chickamauga Creek, which had to be diverted around the dam, were under construction. Twelve bays of the spillway in the second

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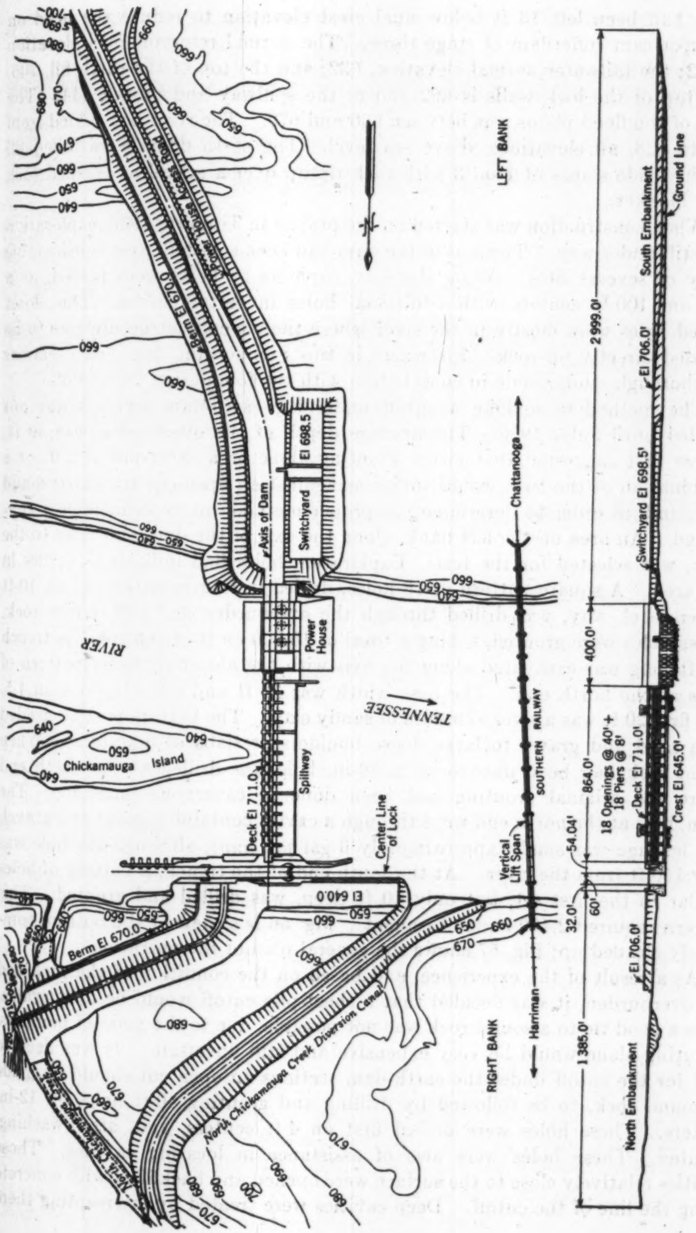
Power House Access Road
Berm El 670.0
Axis of Dam
Switchyard
El 698.5
Power House
RIVER
TENNESSEE
Chickamauga Island
Spillway
Deck El 711.0
Center Line of Lock
El 640.0
Berm El 670.0
North Chickamauga Creek Diversion Canal
North Chickamauga Creek
LEFT BANK
RIGHT BANK
Chattanooga
Harriman
SOUTHERN RAILWAY
Vertical Lift Span
South Embankment
Ground Line
El 706.0
2 999.0'
Switchyard El 698.5'
400.0'
864.0'
18 Openings @ 40' + 18 Piers @ 8'
Deck El 711.0'
Crest El 645.0'
54.04'
60'
32.0'
El 706.0'
1 385.0'
North Embankment
640
650
660
670
680
690
700

FIG. 55.—PLAN AND ELEVATION OF DAM

stage had been left 13 ft below final crest elevation to reduce the head on the upstream cofferdam of stage three. The normal reservoir pool elevation is 682; the tailwater normal elevation, 632; and the top of the earth fill, 706. The top of the lock walls is 692 and of the spillway and intake, 711. The level of the flood plains was between 660 and 670. The river bed varied from 625 to 628, all elevations above sea level. The earth dams were rolled fill and had side slopes of 1 on 3 with rock riprap over a gravel or crushed rock filter blanket.

When construction was started on the project in January, 1936, exploration was still under way. The axis of the dam had been selected after considerable study of several sites. Along the axis, exploratory holes were placed, as a rule, on 100-ft centers with additional holes in certain areas. The closer spaced holes were mostly in the river where the concrete structure was to be founded directly on rock. Elsewhere in this Symposium, Mr. Fox describes the thorough study made in conjunction with the foundation treatment.

The method of making a cutoff under the earth-dam sections was not decided until July, 1936. The average depth of the overburden was 40 ft. It was first suggested that either grouting alone or a sheet-pile cutoff, or a combination of the two, would suffice as against an open-cut trench to sound rock; and, in order to determine the proper method, an experiment was conducted. An area on the left bank, along the axis of the dam and close to the river, was selected for the test. Exploration holes had indicated cavities in this area. A square pattern of 16 holes, drilled in four rows, spaced on 10-ft centers each way, was drilled through the overburden and 100 ft into rock. These holes were grouted, taking a total of 15,000 cu ft of cement. A trench 200 ft long was excavated along the axis with the aforementioned pattern of holes at the north end. The base width was 30 ft and side sloped 1 on 1.5. The first 20 ft was a river alluvium of sandy clay. The bottom portion graded from sand and gravel to large, loose boulders or slabs of limestone. When sound rock had been uncovered, a 36-in. hole was drilled at the north end where the original grouting had been done in cavernous limestone. The 36-in. hole at the north end went through a cavity containing grout and gravel. The leakage was small (approximately 6 gal per min), although the hole was only 150 ft from the river. At the south end of the trench, a pattern of holes similar to the first set, but only 40 ft deep, was drilled and grouted. This pattern required 2,400 cu ft of cement. Fig. 56 is a view of the bottom completely cleaned up; Fig. 57 shows the general plan of the operation.

As a result of the experience, and based on the condition of the rock and the overburden, it was decided that a sheet-pile cutoff would be of no value, since a good tie to a sound rock was not possible, due to the mass of boulders. Grouting alone would be very expensive and still uncertain. It was decided that for the cutoff under the earth-dam sections an open cut should be made to sound rock, to be followed by drilling and grouting 40-ft holes on 12-in. centers. These holes were drilled first on 4-ft centers, and, after washing, grouted. These holes were also of assistance in locating cavities. Those cavities relatively close to the surface were opened and backfilled with concrete along the line of the cutoff. Deep cavities were treated by intercepting them

FIG. 56.—EXPERIMENTAL TRENCH FACING NORTH

with 36-in. drill holes to be used as shafts for cleaning out the cavity and backfilling with concrete. Following this operation, grout holes were drilled midway between those on 4-ft centers. These were washed and grouted. A third set of holes was drilled, washed, and grouted in alternate spaces and, as a final check, the remaining spaces were treated similarly, thus finally spacing the holes on 12-in. centers. Since the bits used on the drill steel averaged about 2.5 in., the rock not actually drilled in the final operation was only about 9.5 in. out of each foot. The system of drilling closely spaced holes on a single line reduced the hazard of missing small cavities which might be the case if holes were staggered over a wide zone. On completion of the grouting, a concrete wall located on the center line and over the grout curtain, 5 ft high with batters of 5 on 1, was built and the trench backfilled with suitable material. Fig. 58 shows the trench ready for backfill.

No further work was done on the south side, or left, bank until the spring of 1937. Since the lock was the first stage of the operations in the river, the next work was done on the earth embankment on the north bank, following the foundation treatment for the lock.

LOCK

The grouting program adopted for all of the concrete structures was in three stages. First, there was a general consolidation grouting (an average of 30 ft deep) of the entire area to be covered, after all loose, cavernous, and seamy rock was removed. The cutoff grout curtain followed, with closely spaced holes to a depth of 40 ft. After concrete had been placed, deep holes along the line of the cutoff curtain were drilled and grouted.

The consolidation system was started with a regular pattern of holes laid out in rows 10 ft on centers and the holes spaced 10 ft in each row. Additional holes were drilled and grouted until all areas were made tight. Re-drilling and grouting was repeated as much as five times in certain areas. Pressures were limited to 30 lb per sq in. Uplift gages were used wherever necessary and further controlled the pressures used. The consolidation grouting required 41,400 lin ft of drilling and consumed 19,300 cu ft of cement.

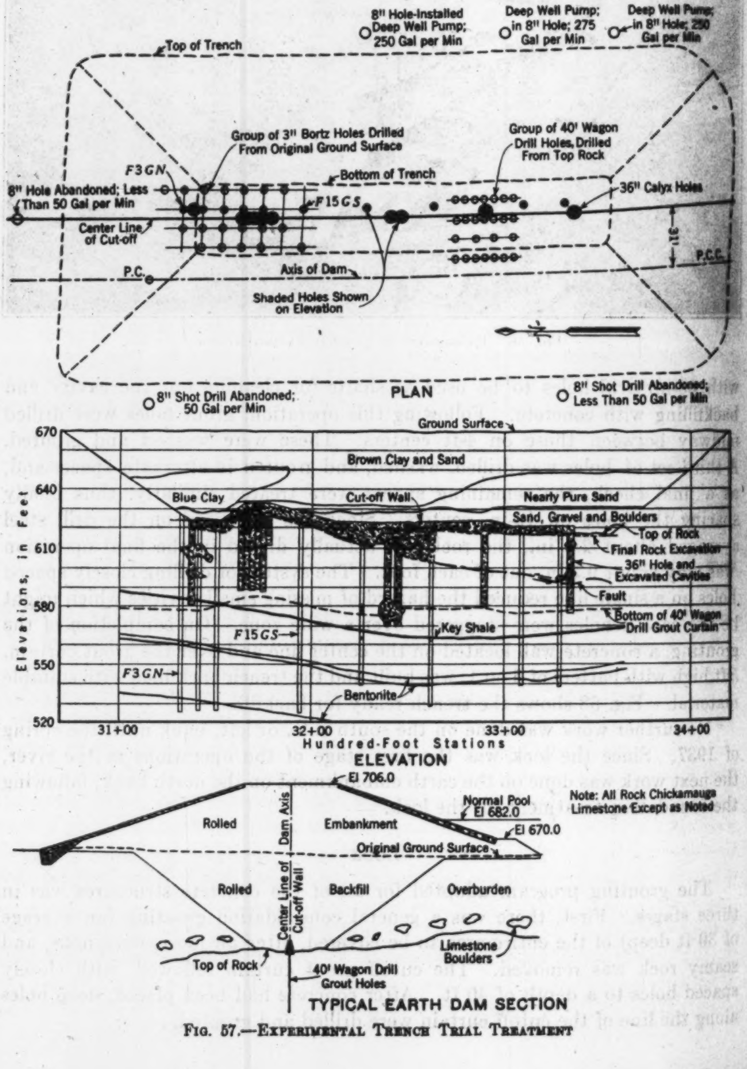
8" Hole-Installed Deep Well Pump; 250 Gal per Min
Deep Well Pump; in 8" Hole; 275 Gal per Min
Deep Well Pump; in 8" Hole; 250 Gal per Min
Top of Trench
Group of 3" Bortz Holes Drilled From Original Ground Surface
Group of 40' Wagon Drill Holes, Drilled From Top Rock
F3GN
Bottom of Trench
36" Calyx Holes
8" Hole Abandoned; Less Than 50 Gal per Min
F15GS
Center Line of Cut-off
31'
P.C.C.
P.C.
Axis of Dam
Shaded Holes Shown on Elevation
8" Shot Drill Abandoned; 50 Gal per Min
PLAN
8" Shot Drill Abandoned; Less Than 50 Gal per Min
670
640
610
580
550
520
Elevations, in Feet
Ground Surface
Brown Clay and Sand
Blue Clay
Cut-off Wall
Nearly Pure Sand
Sand, Gravel and Boulders
Top of Rock
Final Rock Excavation
36" Hole and Excavated Cavities
Fault
Bottom of 40' Wagon Drill Grout Curtain
Key Shale
F15GS
F3GN
Bentonite
31+00
32+00
33+00
34+00
Hundred-Foot Stations
ELEVATION
El 706.0
Dam Axis
Rolled
Embankment
Normal Pool El 682.0
El 670.0
Note: All Rock Chickamauga Limestone Except as Noted
Original Ground Surface
Center Line of Cut-off Wall
Rolled
Backfill
Overburden
Limestone Boulders
Top of Rock
40' Wagon Drill Grout Holes
TYPICAL EARTH DAM SECTION

FIG. 57.—EXPERIMENTAL TRENCH TRIAL TREATMENT

The cutoff curtain under the lock followed and completely surrounded the lock chamber, tying in with the spillway and the north earth dam. These holes were drilled 40 ft deep on 40-ft centers and angled about 20° from the vertical and in the plane of the cutoff. A second set was drilled on the same spacing and slope but angled in the opposite direction. All were washed before grouting. The reason for the different system from that adopted for the cutoff under the earth embankment sections was due to the fact that requirements for sound rock on which to build a concrete structure were more exacting than for the earth dams; hence the close drilling was not necessary. The angle holes would intercept vertical as well as dipping joints, whereas, under the earth dams, the holes were so closely spaced that connections with such joints were generally assured.

FIG. 58.—EXPERIMENTAL TRENCH, FACING SOUTH; READY FOR BACKFILL

In the cutoff grouting, pressures were limited to 50 lb per sq in., and uplift gages were not used. Theoretically, the grouting was being done on a rather narrow band and a proportionately larger volume of rock could be considered as resisting the grout pressures compared to consolidation grouting that was done over a wide area at one time. The consolidation grouting had filled all of the larger and more extensive seams. The cutoff grout curtain required 10,800 lin ft of drilling and took 820 cu ft of cement.

Following the cutoff grouting, a row of diamond-drill holes 60 ft deep was drilled, spaced 20 ft, center to center, along the land wall of the lock from the north earth dam tie-in across the upper miter sill and along the river wall of the lock to the spillway. These holes were drilled and grouted alternately. The total of 69 holes required 1,957 cu ft of cement. To check the secondary holes that required it, a third set was drilled and grouted. However, these

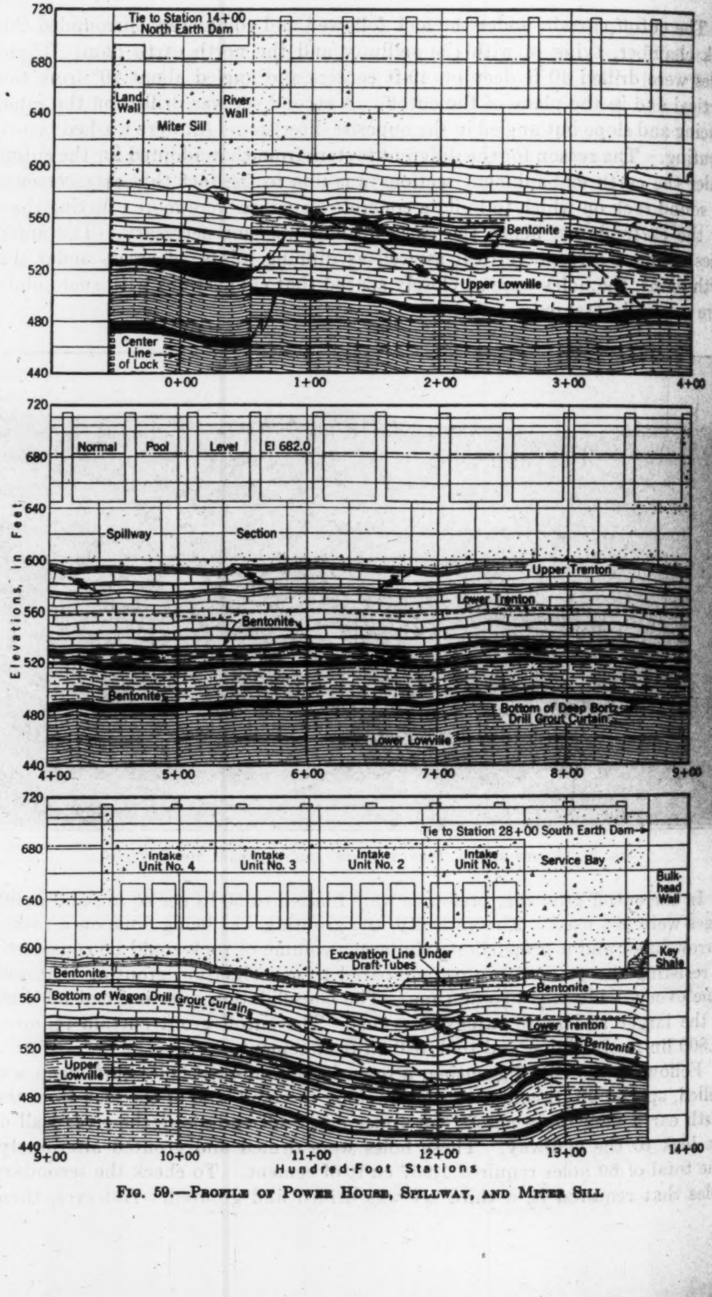
720
680
640
600
560
520
480
440
Tie to Station 14+00
North Earth Dam
Land Wall
River Wall
Miter Sill
Bentonite
Upper Lowville
Center Line of Lock
0+00
1+00
2+00
3+00
4+00
Normal Pool Level El 682.0
Spillway Section
Upper Trenton
Lower Trenton
Bentonite
Bentonite
Bottom of Deep Bort-Drill Grout Curtain
Lower Lowville
4+00
5+00
6+00
7+00
8+00
9+00
Elevations, in Feet
Tie to Station 28+00 South Earth Dam
Intake Unit No. 4
Intake Unit No. 3
Intake Unit No. 2
Intake Unit No. 1
Service Bay
Bulkhead Wall
Key Shale
Excavation Line Under Draft-Tubes
Bentonite
Bentonite
Bottom of Wagon Drill Grout Curtain
Lower Trenton
Bentonite
Upper Lowville
9+00
10+00
11+00
12+00
13+00
14+00
Hundred-Foot Stations

FIG. 59.—PROFILE OF POWER HOUSE, SPILLWAY, AND MITER SILL

latter holes were drilled from 100 to 110 ft deep in order to reach the thick beds of bentonite which carried a small quantity of artesian water. A total footage of 550 lin ft was drilled and 572 cu ft of cement were used. Fig. 59 shows a general outline of the cutoff grouting across the lock, spillway, and power house.

North Earth Dam.—Excavation of the cutoff trench was started at the land wall of the lock, where the conditions were different, at first, from those found in the experimental trench on the opposite bank of the river. For a distance of 500 ft from the lock wall, few loose boulders were found. The rock surface was comparatively level but was crisscrossed with vertical solution channels that followed the dip and strike joint systems in the rock. The width varied from a few inches to 3 or 4 ft, and the depth from a few inches to 8 or 12 ft. Rather than clean out and pour concrete in all these joints, a 10-ft cut in the rock was made with a power shovel. Drilling and grouting in this section developed no serious conditions other than small seams or joints.

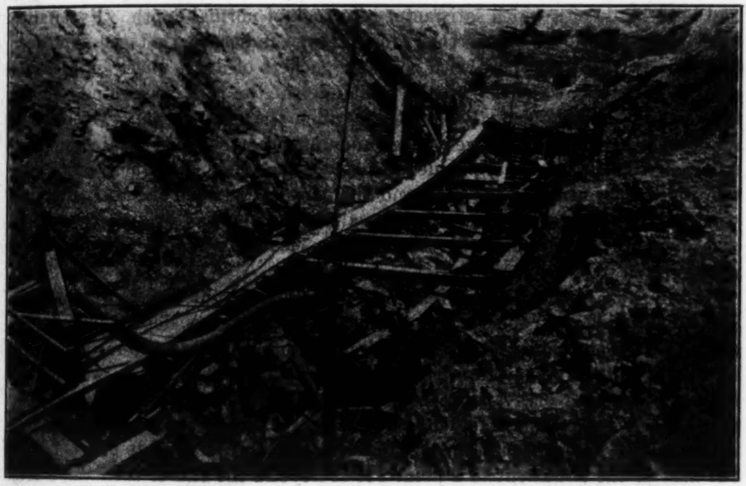

FIG. 60.—SOUTH EARTH DAM FACING TOWARD NORTH ABUTMENT

At 600 ft from the lock wall, a zone of broken rock with numerous mud seams was encountered. A relocation had been made in the axis of the dike and cutoff to avoid a known area where the rock surface was at great depth. This change brought the axis along a line on which, for a short distance, no holes had been drilled directly, although there were a number of them located a short distance on either side.

In the bottom of the trench the rock had been excavated to the point where bracing and cribbing would have been necessary to remove all of the seamy rock, and it was thought that, by repeated drilling, washing, and grouting, the zone could be made tight. This attempt was not successful. Fig. 60 shows this section of the rock. The bottom of the trench in the rock averaged 10 ft in width. It was decided to cover the bottom with a mat of concrete, build the low cutoff wall, partly backfill the trench with clay to increase the stability of the sides of the cut, and then drill 36-in. holes through the mat and excavate the rock by tunneling where the mud seams existed. This operation covered a length of 100 ft along the axis of the cutoff with a general depth of 40 ft. After the tunnel work had been completed, it was backfilled with concrete and grouted. Diamond-drill holes were drilled through this zone and grouted as a final check.

The next 75 ft along the axis was fair rock. Many small seams were found along joints, bedding planes, and two minor faults. In the next 150 ft, a deep depression 65 ft below the normal rock surface was found. Holes close by on either side of the axis gave no indication of its presence. The open-cut trench had been made of a width satisfactory for rock of normal depth. Exploration holes were put down across this zone. These indicated that the depression was filled with a tight clay with zones or pockets of dense chert. Except for clay-filled seams at either end, the remainder of the rock was sound and free from seams or openings. Rather than widen the open trench (which would require the excavation of a very large amount of earth), or excavate and shore a narrow trench to rock, it was decided that a sheet-pile cutoff across this area, followed by grouting of the rock and the contact between the rock and clay, would be satisfactory. At each end, a 36-in. hole was drilled to intercept whatever clay seams existed. These were cleaned out and backfilled with concrete.

From the end of the deep area to the base of the right, or north, abutment, a distance of 340 ft, the rock was excavated to various depths in following mud-filled or clay-filled seams, as they were found. Three 36-in. holes were required to treat deep cavities. The right abutment, which was on the opposite side of North Chickamauga Creek from the dam, was an exposed face of rock that had been folded to a high degree. It was necessary to remove an average of 10 ft of rock to reach material sufficiently sound to grout. From the base of the abutment to a point 350 ft beyond the end of the dam, a total distance of 425 ft, holes from 100 to 175 ft deep had been drilled and grouted on 50-ft centers, followed by holes midway between. In addition, holes penetrating the same zone at a 45° angle were drilled and grouted, using the same spacing and sequence. The bottom of these angle holes covered a zone 350 ft from the base of the abutment. No grouting was done above the elevation of the top of the dam.

In drilling these holes, a fault was found, outcropping at the top of the dam and dipping downward about 30° below horizontal into the abutment. Below the fault, the rock was sound; above it, there were many seams, down to the thick beds of bentonite. Since the outcrop of the fault, as well as the beds of bentonite, was higher beyond the end of the dam, there was no danger of leakage around the end of the abutment.

In addition to the concrete wall normally used, a steel sheet-pile cutoff was built at the abutment. The sheet piling was extended horizontally a distance of 50 ft into the fill from the rock. A steel sheet-pile cutoff, similar

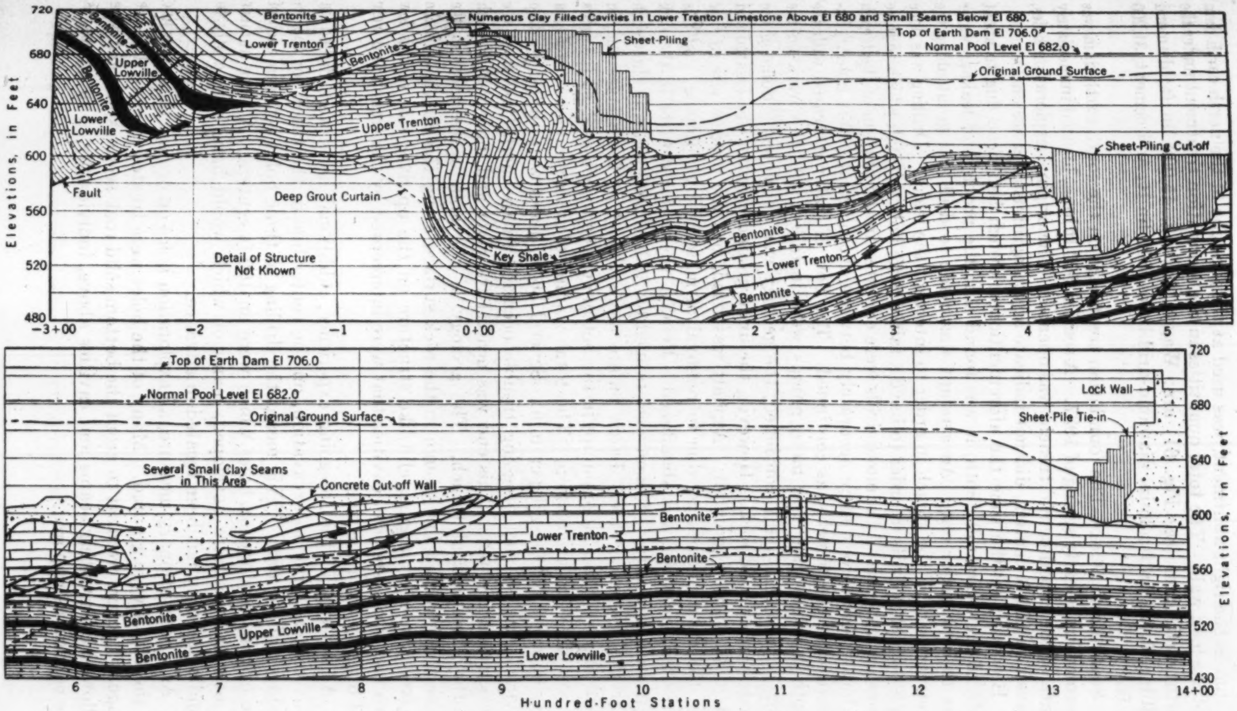
Numerous Clay Filled Cavities in Lower Trenton Limestone Above El 680 and Small Seams Below El 680.
Bentonite
Lower Trenton
Upper Lowville
Lower Lowville
Upper Trenton
Fault
Sheet-Piling
Top of Earth Dam El 706.0
Normal Pool Level El 682.0
Original Ground Surface
Sheet-Piling Cut-off
Deep Grout Curtain
Detail of Structure Not Known
Key Shale
Several Small Clay Seams in This Area
Concrete Cut-off Wall
Lock Wall
Sheet-Pile Tie-in
Elevations, in Feet
720
680
640
600
560
520
480
430
−3+00
−2
−1
0+00
1
2
3
4
5
6
7
8
9
10
11
12
13
14+00
Hundred-Foot Stations

FIG. 61.—PROFILE OF NORTH EARTH DAM

to that in the right abutment, was used at the junction of the north earth dam and the lock wall. The total quantities in the foundation treatment under the north earth dam (see Fig. 61) are: Wagon drilling, 45,000 lin ft; diamond drilling, 6,400 lin ft; 36-in. diamond-drill holes, 500 lin ft; and cement, 31,200 cu ft.

South Earth Dam.—The foundation treatment on the south earth dam was resumed in the early spring of 1937. A section 500 ft long, located immediately south of the experimental trench, was opened. The original exploration holes, spaced on 100-ft centers, did not indicate anything unusually serious.

However, by the time the aforementioned trench had been dug to about 75% of the estimated depth, it was found that the problem of leakage water was to be a major one. An attempt was made to drill and grout along the upstream side of the trench in order to intercept the leaks. Drilling was done with 6-in. shot drills in order to have a fairly large hole through which to force grouting materials. Diamond drills were operated in the bottom of the trench to locate sound rock more accurately between the original 100-ft holes. Excavation of the trench was continued. The flow of water increased with the depth of excavation and more pumps were installed. As the work on the 6-in. holes on the bank continued, the geology was developed. Some of the holes were in solid rock. However, the underground channel was located and attempts made to grout it. Asphalt was tried, but the process was so slow and the cavities so large, due to removal of material by pumping, that this and other grouting was abandoned. Finally, the water courses in the rock were located and two of them were tapped by 36-in. holes drilled in the trench near the upstream side. Large pumps were placed in these holes and, in addition, two other large pumps in the bottom of the trench enabled the crews to complete the excavation. The total volume of water pumped reached a maximum of 15,000 gal per min. Several 36-in. holes were drilled to deep cavities. All of the foregoing trouble occurred in the north 250 ft of the section (see Fig. 62). This end was nearest the river and immediately south of the experimental trench. The geology of the area upstream from the section indicated a low trough in the rock surface along the direction of the strike. An overhanging, cliff-like structure on the up-river side of this low area had two systems of cavities that were intercepted by the trench and were fed directly from the river.

As a result of the foregoing experience, additional holes were drilled on 25-ft centers, prior to excavation, on the center line of the cutoff for the remainder of the dam. This meant the drilling of three holes between each of the original exploration holes which were on 100-ft centers. A large number of new cavities were developed, many of which would have been costly to handle had they not been located in advance.

As a precautionary measure, all exploration holes on or near the center line of the cutoff were grouted. Many of the holes were treated in more than one stage. Packers were set to grout the bottom solid rock sections first and were followed by grouting seams and cavities above, individually or in groups, as the conditions required.

In order to block off, in advance of construction, any water leakage into the trench or cavities, two rows of holes were drilled parallel to the center line, one upstream about 40 ft and another row the same distance downstream. The angle of strike of the rock was taken into consideration in locating these side holes, since the cavities were generally in the same direction. Without washing, these holes were grouted with a low-cost "prescription" grout in which sand, bentonite, and cement were used. The ratios varied somewhat at first but usually 4 parts of sand, 1 part bentonite, and from 2 to 3 parts of cement were used. This mixture cost about half that of straight portland cement grout and was considered sufficiently effective for the purpose. The pressures used were generally 0.5 lb per sq in. for each foot of overburden, or, if in deep rock, 1 lb per sq in. was added for each foot of depth in the rock.

FIG. 62.—CUTOFF TRENCH, SOUTH EARTH DAM, AT POINT OF DEEPEST CAVITY

The bentonite was mostly from Wyoming or the Dakotas. The most suitable grading was: Not less than 90% through 50 mesh; not more than 50% through 100 mesh; and not more than 20% through 200 mesh. This grading was readily mixed, had high swelling powers, and made a good gel.

Bentonite has a dispersion action on the sand and cement, thus keeping all of the materials in suspension. Without the bentonite, the sand would have washed out of the mix and would not have been effective in stopping water. The material is slow-setting and reaches a strength considerably lower than cement grout. The material, when set, is tough and resists erosion to a fair degree. When allowed to dry out, it shrinks and cracks. The bentonite must be mixed with the water before the cement is added; otherwise the dispersing action is mostly lost. The material has a slippery feel until the final set. Another advantage in using bentonite is that it reduces the wear on grout equipment, particularly pumps, when sand is used.

The aforementioned method of preliminary grouting of cavities and solution channels, prior to excavation, was successful in preventing leaks into the excavation. One of the cavities (see Mr. Fox's paper, heading "Description of the Cavities and Weathered Rock: The South Earth Dam and the Left Abutment") extended down to an elevation 90 ft below the river surface and leakage was very small.

In the last 900 lin ft of the cutoff for the south earth dam, no side grouting was done. Throughout this section, all of the seams and cavities were found to be filled with a residual clay. They were not refilled solution channels as were those under the remainder of the structure.

Throughout the south earth dam, the conditions could be stated, in general, as follows (see paper by Mr. Fox for greater detail): In the first 2,000 lin ft south of the river, there were three zones varying from 200 to 400 ft in length that had large cavities and solution channels. The last 1,000 ft, up to the south abutment, had numerous cavities, seams, faults, and joints. Through most of this section the weathered and cavernous rock was excavated from 20 to 40 ft below normal rock surface. The 500-ft section nearest the abutment had to be shored.

The total quantities of cement grout used in the south earth dam were 16,540 cu ft in exploration holes and 70,900 cu ft in the cutoff. In addition, 158,614 cu ft of "prescription" low-cost grout was used, the materials involved being as follows:

Material	Cubic feet
Sand	72,133
Bentonite	14,243
Cement	90,000

In a few holes, sawdust and shavings were added, which totaled 2,238 cu ft, loose measure. A total of 1,500 bbl of asphalt and pitch were used as mentioned previously.

Because of the fact that many, more or less open, cavities were encountered in the cutoff trench under the south earth dam, it was considered advisable to fill them to prevent possible future sinkholes in the rolled fill. Undoubtedly, these cavities were open under adjacent parts of the earth fill. The proposal to fill these cavities with a clay slurry was objected to on the grounds that no pressure could be applied to remove the surplus water from the mixture and thus solidify it. As a result of a series of laboratory tests, F. H. Kellogg, associate materials engineer, found that a local river terrace deposit of red clay could be stabilized to form a satisfactory grout by the addition of portland cement. This clay had a chemical analysis (based on samples dried at 110° C) as follows:

Compound	Percentages
Silica (SiO_2)	62.83
Alumina (Al_2O_3)	18.26
Iron oxide (Fe_2O_3)	7.50
Lime (CaO)	None
Magnesia (MgO)	0.28

Compound	Percentages
Sulfuric anhydride	None
Phosphorus pentoxide (F_2O_5)	0.18
Titanium dioxide (TiO_2)	1.45
Manganous oxide (MnO)	0.028
Sulfide sulfur, as FeS_2	None
Loss on ignition, 110° C–1,000° C	7.16
Insoluble residue	82.68
Total sulfur (S)	None
Potassium oxide (K_2O)	0.61
Sodium oxide (Na_2O)	0.02

The sieve analysis was 25% clay (− 0.005 mm), 47% silt (+ 0.005 mm and − 0.05 mm), and 28% sand (+ 0.05 mm). This clay is of the type that crumbles on drying, which makes it suitable for mixing and handling in most types of grout equipment. This stabilized clay grout is mixed in the proportion of 7 cu ft of clay and 1.5 cu ft of cement. The total water amounts to 6 cu ft, including that already in the clay. This material reaches an initial set in 11.5 hr and a final set in five days. Tests show the shrinkage to be practically nothing. The shear strength is 34.8 lb per sq in. Cored samples of clay grout have been obtained with both shot and diamond drills, indicating that the material has hardened satisfactorily.

Two field tests were made which further indicated that this grout would be satisfactory. Two 50-gal drums were filled with water. One was also filled with crushed rock, 1.5 in. to 3 in. in size. A grout pipe was inserted so that the grout was introduced at the bottom and the water was displaced from the bottom upward. When all of the water had been displaced, the grout pipe was removed and the material allowed to set. The remaining drum was filled with grout by allowing the mixture to drop through the water until the drum was full of grout. The object was to determine whether the cement would separate from the mixture under such conditions. Practically all of the open cavities had water in them and it was important to know about any possible separation of cement from the clay.

Both of these samples were allowed to set for one week, when the drums were out and the cylinders of grout removed. In both cases the grout was hard, except for a few inches at the top. The samples were left exposed to the atmosphere, heavy rains, and long dry spells and, after six months, they were still in excellent condition until freezing weather, when the sample that had been poured directly in the tank of water began to spall. These samples were deposited under no pressure. Under regular grouting operations where pressure is applied, any surplus water is forced out, thus further solidifying the mix.

This material was selected for filling cavities under the earth dam not otherwise treated in the cutoff. By observation in the cutoff trench, and from exploration holes, the trend of the cavities was established. Many seams and openings existed in the rock side walls of the core trench which could not be filled with clay in the backfilling operation.

In all such cases, grout pipes were inserted and brought up as the fill progressed. As soon as the fill reached the natural ground line, these pipes were grouted with the stabilized clay grout.

In the section where there had been heavy pumping, it was necessary to place large casings, for pump sumps, as the fill progressed. Corrugated culvert pipe was used for casing. When the fill had been completed to a point above the ground-water table, that was the same as the river surface, the pumps were removed. Grout pipes were installed, reaching to the bottom. The casings were filled nearly to the top with coarse rock. A concrete cap, 1 ft thick, was placed over the rock in each casing. Two vent pipes extended through each cap. The casings were filled with clay grout.

In addition to filling cavities under the earth dam, the area upstream from the section where the heavy pumping was required was filled with clay grout. This was done for two reasons. One was to eliminate any settlement immediately upstream from the toe of the dam and the other was to prevent leakage around the land connection of the upstream area of the third-stage cofferdam. In this area, a total of 36,000 cu ft of clay grout was used, in addition to 37,996 cu ft of sand-bentonite-cement grout.

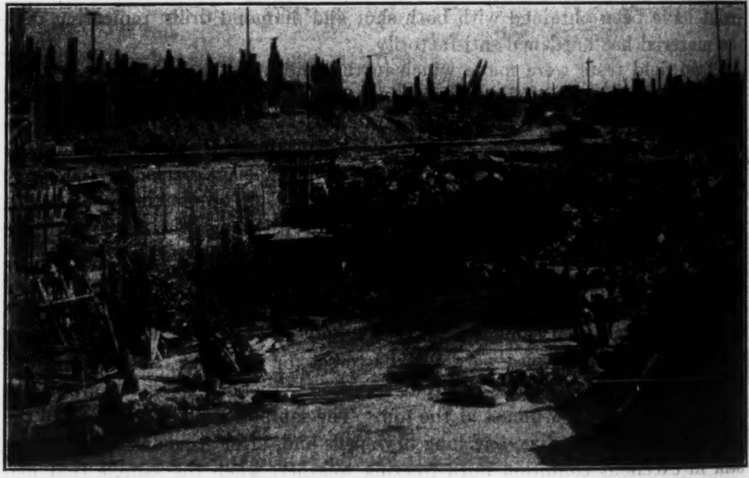

Fig. 63.—Spillway Excavation

Spillway and Power House.—The excavation of the spillway and part of the power-house sections was greatly simplified by the occurrence of the shale and bentonite which was the key bed used in tracing the geologic structure over most of this area. Above this bed were numerous solution channels and cavities. Where this bed was covered with a reasonable layer of rock and where no faults or sharp folds existed, it was the rule rather than the exception that there were no cavities, solution channels, or weathered seams below. As a result, the consumption of grout under such conditions was very low. Fig. 63 shows excavation to the key bed of shale and bentonite in the spillway area. The smooth rock floor is clearly visible. Solution channels along the upstream face, at the left, have been walled up with concrete, the forms being still in place.

A general consolidation grouting program was developed with 30-ft wagon-drill holes on 10-ft centers. However, the rock was found to be sound and required so little grout that the hole spacing was changed to 12 ft and later to 20 ft. The total cement used for spillway and power-house consolidation grouting was 3,050 cu ft in 672 holes. Uplift gages were used when it was considered necessary.

For the cutoff grouting, the holes were drilled 4 ft downstream from the upstream face in two sets with both in the same vertical plane. The first set was drilled 40 ft at an angle of 25° in one direction. These were washed and grouted. The second set was drilled as a check but inclined in the opposite direction. These holes required relatively little grout. Many of the consolidation and cutoff holes scarcely took enough grout to fill them. The total quantity of cement used in the cutoff was 530 bags in 370 holes.

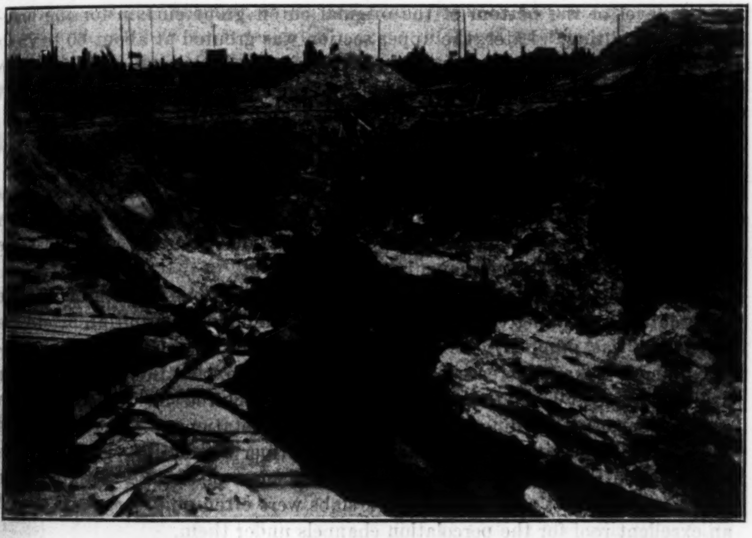

Fig. 64.—Power-House Excavation, Facing Upstream; Deep Solution Channel Opened Up Along Fault

A large part of the power-house foundation was complicated by the occurrence of a rather flat fault, the outcrop in the final foundation extending almost diagonally across the downstream corner of the service bay at the south end to the upstream end of Unit No. 3. This fault dipped in an upstream direction. The service bay was excavated 30 ft deeper than would ordinarily have been required, due to the cavernous condition of the rock. This held true for the intake structure, but to a lesser extent. Fig. 64 shows the cavernous nature of the rock under the intake. After the final bottom for excavation had been reached, a few mud seams were found in and above the fault for a distance of 25 ft from the outcrop. A part of this was excavated and the remainder was treated by repeating the washing and grouting several times.

In addition to the cutoff grouting with the wagon drills, deep holes, spaced 12 ft on centers, were drilled along the same cutoff line with a diamond drill, after the concrete was placed. The object of these holes was to penetrate the two thick layers of bentonite. All holes penetrated the upper layer, and the alternate holes were drilled through the lower bed. These holes were drilled alternately, the deep ones being treated first. No washing was done on these holes, since there were no mud seams. Higher pressures were used, to 125 lb per sq in. in the lower portions of these holes. Packers were set at the level of the bottom of the original cutoff grout curtain for the high-pressure grouting. Later, the upper section was grouted at about 60 to 75 lb pressure. The total quantity of grout used in the high-pressure holes in the spillway and power house was 1,100 cu ft of cement in 114 holes, which totaled 11,500 lin ft. In the entire power house and spillway, including cutoff, high-pressure, and consolidation grouting, 12,207 cu ft of cement were injected in 1,231 holes.

Temporary Construction Grouting.—The preliminary grouting on lines parallel to the cutoff trench under the south earth dam, for convenience in construction, has been mentioned previously. This grouting also had some beneficial effect on the permanent structure.

In addition, considerable cofferdam grouting was done. In the first-stage cofferdam surrounding the lock, there was no preliminary grouting before the cofferdam was pumped. As excavation progressed, particularly in the upstream end, leakage increased. A few preliminary exploration holes had been drilled previously to indicate the conditions on which the sheet-steel cofferdam was designed. According to the original interpretations, alternate layers of rock and filled cavities were found below the river bed of gravels. However, it developed that this section was a confused mass of clay, gravel, and large boulders or slabs of rock. These rock slabs were often quite large and offered an excellent roof for the percolation channels under them.

Grouting was done during the time the cofferdam area was unwatered, and after leakage had become serious. Holes were drilled inside of the cofferdam, extending from the upstream land connection to a point about one third of the way along the river arm. Most of the drilling was done with 5.5-in. shot drills. A large hole was considered advisable in order to force coarse materials through where necessary. Holes were tested with dye to determine what connections existed. Various types of grout were used. Sand, bentonite, and cement mixtures were used in many of the holes. To this mix, sawdust and shavings were added and even strips of burlap and small cloth sacks of bentonite. Asphalt and pitch were used also. From some of the holes, this material leaked into the springs in the excavated areas. Accordingly, a wide berm was built outside of the cofferdam cells; and clay, cinders, and hay were sluiced in, or placed over, areas in the river bed and nearby creek channel, where sacks, dragged along the bottom, indicated that leakage might have started. Eventually, the leakage was reduced and the construction of the lock completed, but not without considerable trouble.

The total quantities of each type of grout used in grouting the first-stage cofferdam were as follows:

Item	Material	Cubic feet
1	Straight cement	1,480
2	Sand-bentonite-cement	19,900
3	Sawdust, etc., with Items 1 and 2	3,730
4	Asphalt and pitch	36,350

The piers supporting the upstream extension of the river wall of the lock were designed to be built directly on rock. Conditions similar to the foregoing, under the adjacent section of the cofferdam, prevented open excavation; and the pier footings were redesigned, using reinforced concrete columns. This difficult operation also required much grouting to check inflows of water as holes were driven for the columns. These holes varied in depth from 40 to 80 ft.[17]

As a result of the experience in the first-stage cofferdam, a thorough geological study was made of the conditions under each of the two following stages prior to their installation. Additional holes had been drilled and the geologists were able to make an accurate interpretation of the foundation.

The decision was made to grout such areas as might develop leaks in advance of pumping out the cofferdam. The studies indicated that in both stages the upstream arms would require this treatment. Cofferdam No. 2, which surrounded most of the spillway, was grouted along the upstream and river arms. Geological studies indicated that grouting of the downstream arm would not be necessary.

In order to expedite the grouting and not delay pumping out, this work was started as soon as possible. In each cell, two 6-in. casings were set and braced to the steel piling prior to filling the cells with gravel. In each diaphragm connecting cell, one casing was installed, making the average spacing about 17 ft, center to center. This line of holes was located near the downstream side of the cells. Holes were drilled as fast as the cells were filled, and they were grouted as rapidly as possible. Alternate holes were drilled and grouted first to keep each operation from interfering with the other. During grouting, a close watch had to be maintained for direct leaks in the river. Where these leaks were found, grouting would be slowed up to allow the leak to plug. Frequently, calcium chloride was added to speed the setting; sawdust was also added. A limit of 500 sacks of cement per hole was established to avoid unnecessary waste.

The cofferdam was completed and pumped out before any regrouting could be done. Additional drilling was done inside of the cofferdam and adjacent to the cells, which avoided the additional expense of drilling from the tops of the cells, to check up on sections where the first grouting indicated that further treatment was necessary. This explains the reason for locating the casings in the cofferdam cells on the downstream side or inside face. In this

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[17] *Proceedings*, Am. Soc. C. E., June, 1939, p. 953.

Method of Operation.—All diamond-drill holes were logged carefully by an inspector. The notes and the cores were studied by a geologist. Wagon-drill holes were watched, during drilling, for evidence of mud seams. A feeler was used in searching for small seams and to check drilling data.

Before grouting, the holes were washed in groups. Notes were made on the characteristics of each hole so that when grouting was started the inspector and engineer in charge had all the available information on the underground conditions upon which to base the method of starting and conducting operations. Accurate records were kept during the grouting of each hole so that an intelligent interpretation could be made of the results, and to determine the sufficiency, or the next method of attack.

Rigid instructions for grouting were not given; instead, flexible rules for guidance were used, since all holes do not act alike. With information available as to how a hole or group of holes acted during washing operations, grouting was started with a mix as thick as it was considered would be taken freely at a pressure of about 75% of the limit allowed. From this point on, the inspector would vary the water-cement ratio by thinning it if the pressure went up and thickening it if it dropped. Under the concrete structures, uplift had to be watched and considered in determining the mix and pressure. Grout mixes varied in consistency from 5.0 to 0.6 water-cement ratio, measured by volume.

CONCLUSIONS

The foundation treatment at Chickamauga Dam falls into three divisions: (1) That under the earth-dam sections; (2) that under the concrete structures; and (3) that of construction, or temporary grouting. Realizing that nothing but the most thorough job would do, it became a question of devising the most economical method.

Under the earth-dam sections, it was necessary to balance the cost of excavation, plus the earth backfill, against the probable cost of grouting, and dig to the point where the sum of the two was the lowest. There were many locations where a grout crew could have spent money at the rate of $2,000 to $3,000 per day on an individual hole had this not been anticipated. As it was, a much smaller total sum was spent by excavating to the point where grout consumption was relatively small. The method of excavation, whether by open cut or by shafts, depended on the depth. The foundation for the earth dams was excavated, not with the idea of placing a concrete structure on it, but to secure a low-cost cutoff. The attempt was made to keep the grout in the line of the cutoff curtain and not over a wide zone. The sequence of drilling, washing, and grouting was decided with this end in view. If the same number of holes had been drilled and grouted over a wide zone, the cutoff line would not have been as tight as the method followed. It could be compared to building a picket fence. In the case of the wide zone, four fences could be established with the pickets wide apart; whereas, if all the pickets were built into a single fence, a tighter barrier would be established.

The conditions under the concrete structures, lock, spillway, and power house were different. A firmer rock was required. The occurrence of bentonite beds, with sound unweathered rock below, together with the depth required for various portions of the structure, determined the limits of excavation under all of the concrete structures, with the exception of the upstream end of the lock and the south half of the power house.

In the case of the lock guide walls, where the hydraulic head would always be balanced, the requirements for the foundation were not as exacting as other parts of the structure. In the consolidation grouting that was applied to the entire foundation under concrete structures, more grout was used under the lock guide walls than elsewhere. This reduced the excavation and speeded the progress of the job as a whole. It was a question of balancing the cost of excavation (plus the concrete to fill it up) against the grouting costs, and still meet the requirements for soundness.

The necessity for temporary construction grouting was a result of experience gained in the first cofferdam, around the lock area, and in the cutoff trench excavation on the south side. The requirement was for a low-cost grout that was needed for temporary purposes only, since a large volume was anticipated. Many different materials were considered but the high costs ruled most of them out. The sand-cement-bentonite grout cost less than half that of straight cement. This combination mixture proved satisfactory. The stabilized cement-clay grout used for backfilling cavities under the earth dams is far more permanent than any material already existing in them, and the cost was one third less than the sand-cement-bentonite mix.

In conclusion, it is recommended that, for any dam site situated with similar foundation characteristics, an exploratory drilling program be planned with closely spaced holes. At Chickamauga Dam, the preliminary exploratory information was of little value until holes were drilled on 25-ft centers. Careful drilling must be done in order to recover as complete a core as possible. Accurate logging of holes is necessary.

ACKNOWLEDGMENT

H. E. Murray, associate engineer, supervised the field grouting operations under the writer's direction and assisted the writer in preparing this paper.

DISCUSSION

George K. Leonard,[19] **M. Am. Soc. C. E. (by letter).**—The foundation conditions at Guntersville Dam, and the treatment of the rock to make it watertight, have been described accurately and completely in the papers by Mr. Gongwer and Mr. Ross. Few dam foundations in limestone are found that are better than that under Guntersville Dam, unless one can imagine finding limestone without solution channels. The engineers of the U. S. War Department who did the first drilling on the site in 1914, and the TVA engineers who completed it and made the final decision on the dam location, are to be complimented.

Although the exact location of the dam was not dictated by the location and height of Wheeler Dam (the next dam downstream), the general location was fixed within the limits of several miles. Within these limits several dam sites could have been found; and, in fact, several were investigated which could have been developed satisfactorily.

As has been described, the bedding of the limestone was tight and of ample thickness. It was nearly horizontal but what slight dip there was, was in the right direction (upstream). In the entire length of 4,000 ft, only two areas totaling about 600 ft in length showed signs of being cavernous or unsound; and these areas were fairly close to the surface.

The writer has often wondered whether the locating engineers knew exactly how good a site they were choosing or whether the significance of the formation that made it a supersite was unknown to them. Even if the latter was the case, they still deserve much credit for their excellent choice. It would have been a good site without the super feature and by reason of its presence few better limestone foundations can be imagined.

The feature that changed the site from an excellent to a nearly perfect one is the continuous, insoluble, shale horizon which underlies the entire site. After the engineers had fully realized its significance, no further uneasiness was felt about the foundation. They knew that below the shale no solution had taken place and that the rock must be sound and tight. Above the shale they knew that they might find anything that is usually found in limestone bedding; and, indeed, in one place or another, throughout the length of the dam, they actually found about everything.

The shale seam outcropped a short distance below the dam, and it is possible that the relocation of the dam downstream to a point nearer the outcropping would have found the shale seam still closer to the surface, thus saving some excavation and further improving the foundation. Although hindsight is always better than foresight, this possibility cannot be stated as a certainty.

The shale along the axis varied in depth from 15 to only 25 ft, so the cost would not have been excessive even had it been found necessary to make a concrete cutoff through the uncertain rock above the shale for the entire length of the dam. Had this been done, grouting might have been dispensed with in the rock below the shale, with perfect safety, although this would have been contrary to orthodox engineering practice. Since the rock above the shale was fairly sound in most places, the economical method of treatment consisted in general of removing all disintegrated rock and saving as much of the sound rock as possible, treating it in various ways to consolidate it.

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[19] Constr. Engr., TVA, Cherokee Dam, Jefferson City, Tenn.

Grouting procedure was not difficult or unique; but the search for, and filling of, caves under the south embankment disclosed many weird geologic formations and presented a series of problems which Mr. Gongwer has described. The control of underground water in the confined space between the sheet-pile walls was difficult, but in no place except the final closure was concrete placed in water. It is believed that the cofferdam method of opening up the cutoff is superior to the open-cut method, not alone because the cost is less but because the contact of the cutoff piling with the rock or concrete core wall can be seen before backfilling the trench. The open-cut must be backfilled before driving the piling, and it is impossible to see the seating of the piles on the uneven rock. Some engineers might think the piles were unnecessary if the trench was backfilled with impervious clay. This might be true, but it would depend upon the nature of the original overburden. If this was fairly pervious, the clay core at the bottom might be too thin. Of course, a level concrete foundation could be placed on the rock on which to seat the piles.

Grouting of the disintegrated rock under the cofferdam cells is believed to be a new technique for cofferdam construction. Cellular type cofferdams can be built safely on solid rock or on soft homogeneous material such as sand. In the latter, seepage is high and berms must be wide enough to prevent piping. Systems of well points and large pumping capacity are necessary to keep the excavation dry.

On rock bottoms it is customary to build the cofferdam as close to the line of excavation as working conditions and plant layout will permit. If open or gravel-filled solution channels or mud seams are exposed on the face of the cut, trouble begins immediately. This condition developed in the lock cofferdam and caused considerable expense and worry. A construction man will fight a leak to the bitter end with everything at his command rather than allow the dam to flood after it is once unwatered. This is what the construction forces did, but they finally lost. Looking backward again it would have been much cheaper, and no more time would have been consumed, if the dam had been flooded when the first sign of disintegrated rock appeared and if it had then been grouted under equalized head.

The experience gained in the lock cofferdam was used to advantage in the spillway and power-house cofferdams. Here, where the piling in the cells landed on unsound rock, grouting was completed before unwatering. In each of these two large areas of about 6 and 9 acres, respectively, the seepage was not enough to tax a 6-in. pump. The cofferdam grouting gave the work all the advantages of a dry-land location. Certainly no more justification is necessary.

The importance of visual inspection of foundation rock, in cored holes large enough for the inspector to be lowered into them, cannot be overemphasized. Logs of, and cores from, small cored holes tell many things; but they do not tell enough, and are likely to lead to erroneous deductions and conclusions. Logs of holes made with percussion drills are almost worthless unless the rock is either very poor or very good.

How much preliminary exploratory drilling should be done before construction starts? Obviously, this depends on the kind of rock, its condition, the information desired, and many other things; but in general, the writer doubts the economy of attempting to show all subsurface conditions completely with preliminary drilling. Only general conditions should be investigated, leaving the gaps to be filled in by the construction forces as the work progresses. Contract work and force account work would require different plans for exploration. Limestone like that at Guntersville, and breccia like that at Boulder Dam, would require different investigation.

One can well imagine the difficulty of making a complete preliminary investigation at either the Chickamauga or Guntersville sites upon which a contractor could safely bid and about which there would be no argument; yet, when done by force account, the corrective treatment has been expeditious and complete.

F. B. Marsh,[20] M. Am. Soc. C. E. (by letter).—The experiences described in this Symposium are particularly interesting to any one who has tried to shut off underground flow under a dam or other structure. The work accomplished by the TVA in this direction seems to have been exceptionally thorough and effective.

Nevertheless the question inevitably arises whether a dam founded on a soluble limestone can ever be made entirely safe as a long-term proposition. This is especially true when one considers the normal tendency of maintenance organizations to confine their attention, after construction organizations are gone, to strictly necessary routine matters and to give no thought to gradual changes that may be occurring, such (for example) as the slow development of springs downstream from a dam or from a low divide near the rim of a reservoir.

Even if the dissolving action may be very slow, as noted by Mr. Fox, the ultimate effectiveness of ground water as a solvent of limestone is clearly demonstrated by the conditions in this Tennessee valley, not to mention the cave regions in the Shenandoah Valley of Virginia and elsewhere. It is not difficult to imagine that a leak, now so small as to be unnoticed, in a long term of years may develop into something dangerous, regardless of the cutoffs so painstakingly provided, as described in this Symposium.

Such considerations make it advisable to sound a warning to other less careful and perhaps less permanent organizations than TVA, which may be tempted to build dams on limestone foundations, that they are treading on dangerous ground—on ground that can be made safe only at great expense. One of the writer's recollections of younger days is the acrimonious discussion that followed the failure of the Colorado River dam near Austin, Tex., in 1893, after only eight years of service, the limestone foundation being one of the factors involved.[21]

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[20] Div. Engr., Eastern Dept., New York Board of Water Supply, White Plains, N. Y.

Berlen C. Moneymaker,[22] Assoc. M. Am. Soc. C. E. (by letter).—The site of Chickamauga Dam on the Tennessee River just upstream from Chattanooga is of particular interest to both the engineer and the geologist because of the difficult foundation problems it presented. The choice of the dam site was limited to a relatively short section of the river which did not afford a wholly satisfactory location. Two sites had been drilled by the Corps of Engineers, U. S. Army, before the Tennessee Valley Authority (TVA) was created, and four additional sites were subsequently explored by the Authority. Although the site finally chosen was obviously the best one available, certain defects of a geologic character were recognized from the outset.

As Mr. Fox has pointed out, Chickamauga Dam is located on the great variety of thin-bedded limestones with thin, interstratified layers of bentonite and shale which are known collectively as the Chickamauga limestone. The Chickamauga limestone is in fault contact with the Knox dolomite about 3,250 ft upstream from the dam. This major thrust fault represents a stratigraphic displacement of about 5,000 ft, and as the Knox dolomite was thrust over the Chickamauga limestone, both formations are deformed for a considerable distance on either side of the fault. Many very sharp folds and even a greater number of faults occur in the rocks on which the various parts of the dam and lock are located. There were no outcrops of bedrock except along North Chickamauga Creek, at the extreme north end of the dam, where several very sharp, asymmetrical anticlines and synclines were exposed. In the river channel and flood plains, rock was covered by alluvium (clay, silt, sand, and gravel), which was more than 50 ft thick in some places. There was a thin, discontinuous cover of residual clay at the extreme north end of the dam, and both residual materials (mostly clay) and terrace gravel overlie bedrock at the extreme south end of the structure.

Because of the very complex geologic structure and the great thickness of of the overburden, the geologic conditions were very difficult to determine. Even after several thousand feet of drilling was done, little was known definitely except that the upper portion of bedrock was generally quite cavernous. At this stage, Mr. Fox was assigned to the Chickamauga project as the resident geologist. By very careful and painstaking study of the cores recovered from the drill holes, he was able to discover a very thin layer of shale and bentonite sufficiently different from all other rocks to be recognized wherever found. The value of this layer as a "key" bed was immediately appreciated. After this discovery was made, the cores recovered from all previously drilled holes were restudied in detail, and within a very short time all but the more minute details of geologic structure were developed. At the same time that the key bed was discovered, it was also discovered that, although the rock above it was everywhere quite cavernous, the rock below it was entirely free from cavities except along a fault or two. This made possible the very accurate prediction of the elevation, at any point in the dam site, below which the rock was free from cavities.

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[21] *Engineering News*, Vol. 29, June 8, 1893, p. 545, and June 29, 1893, p. 618; also, *loc. cit.*, Vol. 30, July 27, 1893, p. 78.

[22] Senior Geologist, TVA, Chattanooga, Tenn.

The cavities constituted the most perplexing and the most serious of the foundation difficulties. They were developed mainly along joints and faults, although some of the larger ones on the south (left) side of the river exhibited a marked tendency to follow certain of the more soluble beds. On the whole, the cavities were very irregular in form. Some of them were continuous for appreciable distances both laterally and vertically. A cavity at station 55/24 extended downward to El. 510, a few feet below the level of the Wheeler Reservoir. Hundreds of small cavities were encountered throughout the dam site at various elevations both above and below the water table. The incorrectness of the concept long held by geologists that cavities may not be formed below the water table is nowhere better demonstrated than at Chickamauga. At this locality, the depth to which cavities were formed was determined, not by the position of the water table, but by the position of a layer of bentonite and shale, designated as the key bed, which effectively shielded the pure, underlying limestone from solution.

The early recognition of the seriousness of the foundation conditions at Chickamauga Dam was very fortunate. By taking great care in the exploration, preparation, and treatment of the foundation, a safe dam has been constructed on a very unsatisfactory site.

In his excellent paper, Mr. Lewis presents an interesting and useful record of the foundation treatment and rim tightening work done by the TVA at Norris Dam. Although the paper treats all phases of the work, this discussion is restricted to those bearing upon foundation conditions.

As might have been pointed out by Mr. Lewis, the site of Norris Dam was selected by the U. S. Corps of Engineers previous to the creation of the TVA. A considerable amount of preliminary investigation and exploration, on the basis of which the feasibility of the site was determined, was done by the Corps of Engineers. Further exploration by the TVA did not materially alter the picture of foundation conditions afforded by the previous work.

The entire dam site is in approximately the middle third portion of the Copper Ridge (lower Knox dolomite) which was designated as the Rockhouse member by Arthur Keith.[23] The formation is about 1,000 ft thick at Norris Dam, and consists mainly of cherty dolomite and cherty magnesian limestone. As a whole, the formation is quite massive in character, the individual strata ranging in thickness from a few inches to about 20 ft. The strata dip rather uniformly downstream (southeast) at an average inclination of 4°, the strike being N. 65° E.—approximately parallel to the axis of the dam. Stratification planes were well marked throughout the formation. In the upper portion of bedrock, contiguous strata were usually found to be separated by thin layers or seams of residual clay. At greater depths, however, the strata were usually in close contact, although separated by a distinct break. In nearly all cases, these planes were sufficiently open to be water bearing, or potentially water bearing.

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[23] "Geology of Norris Dam Site," by Arthur Keith. Unpublished report to TVA, August 30, 1933, p. 7.

The rock involved in the dam site was entirely free from faults, although it was extensively jointed. Most of the joints were merely incipient structures, along which the rock broke on blasting, but some of them were fissures, commonly somewhat enlarged by solution. Some of the joints thus enlarged were open, but in most cases they were more or less completely filled with residual clay. Joints exhibited a great diversity of trends, but most of them were characterized by northeast trends and steep dips.

As is almost always the case in limestone and dolomite, the rock forming the foundation, abutments, and adjacent portions of the reservoir rim was, to some degree, cavernous. This condition was clearly recognized from the start, and construction was not undertaken until its seriousness had been fully ascertained. With the exception of a cave—6 to 10 ft wide and 10 to 25 ft high—in the right (west) abutment at El. 980, the openings resulting from solution of the rock were found to be of relatively small dimensions, except for lateral extent. Cavities were appreciably larger and somewhat more abundant in the abutments than in the foundation. Drilling revealed that cavities and seams ranging in vertical dimensions from less than 0.1 to 3.0 ft occurred in the foundation to a maximum depth of 86 ft below the original bed of the river. These cavities averaged slightly more than 0.3 ft in vertical thickness, and most of them were filled or partly filled with residual clay. Nearly all of the small cavities encountered in the foundation seemed to represent merely enlarged stratification planes, but the larger ones in the abutments were, for the most part, developed along joints. Some strata were found to be decidedly more cavernous than others because of their greater susceptibility to solution.

The cavities were not sufficiently large, open, or closely spaced in either the abutments or foundation to render the rock incompetent to bear the load to be imparted by the structure. Nevertheless, they constituted serious leakage hazards and made necessary the extensive and systematic treatment with which Mr. Lewis' paper deals. Without such remedial treatment, leakage undoubtedly would have occurred as soon as the reservoir began to fill. As leakage progressed, the clay filling would have been washed out of the cavities and seams, resulting in the loss of increasing volumes of water. The cost and difficulty of stopping, or even appreciably retarding, leakage that has developed in limestones or dolomites under and around a large dam can be thoroughly appreciated only by one who has attempted such a task.

The systematic washing of clay out of seams—most of which were along bedding planes and consequently inclined downstream at an average of 4°—and refilling the openings with grout removed the very slight possibility of failure by sliding within the foundation and the probability of settlement under the load of the dam, as well as leakage hazards.

The advantageous use of large-diameter drill holes in foundation exploration and foundation treatment is justifiably emphasized by Mr. Lewis. Norris is one of the first large dams in the United States at which large-diameter core drilling was done on a moderately large scale for both exploration and foundation treatment.

everywhere quite cavernous, the rock below it was entirely free from cavities except along a fault or two. This made possible the very accurate prediction of the elevation, at any point in the dam site, below which the rock was free from cavities.

The cavities constituted the most perplexing and the most serious of the foundation difficulties. They were developed mainly along joints and faults, although some of the larger ones on the south (left) side of the river exhibited a marked tendency to follow certain of the more soluble beds. On the whole, the cavities were very irregular in form. Some of them were continuous for appreciable distances both laterally and vertically. A cavity at station 55/24 extended downward to El. 510, a few feet below the level of the Wheeler Reservoir. Hundreds of small cavities were encountered throughout the dam site at various elevations both above and below the water table. The incorrectness of the concept long held by geologists that cavities may not be formed below the water table is nowhere better demonstrated than at Chickamauga. At this locality, the depth to which cavities were formed was determined, not by the position of the water table, but by the position of a layer of bentonite and shale, designated as the key bed, which effectively shielded the pure, underlying limestone from solution.

The early recognition of the seriousness of the foundation conditions at Chickamauga Dam was very fortunate. By taking great care in the exploration, preparation, and treatment of the foundation, a safe dam has been constructed on a very unsatisfactory site.

In his excellent paper, Mr. Lewis presents an interesting and useful record of the foundation treatment and rim tightening work done by the TVA at Norris Dam. Although the paper treats all phases of the work, this discussion is restricted to those bearing upon foundation conditions.

As might have been pointed out by Mr. Lewis, the site of Norris Dam was selected by the U. S. Corps of Engineers previous to the creation of the TVA. A considerable amount of preliminary investigation and exploration, on the basis of which the feasibility of the site was determined, was done by the Corps of Engineers. Further exploration by the TVA did not materially alter the picture of foundation conditions afforded by the previous work.

The entire dam site is in approximately the middle third portion of the Copper Ridge (lower Knox dolomite) which was designated as the Rockhouse member by Arthur Keith.[23] The formation is about 1,000 ft thick at Norris Dam, and consists mainly of cherty dolomite and cherty magnesian limestone. As a whole, the formation is quite massive in character, the individual strata ranging in thickness from a few inches to about 20 ft. The strata dip rather uniformly downstream (southeast) at an average inclination of 4°, the strike being N. 65° E.—approximately parallel to the axis of the dam. Stratification planes were well marked throughout the formation. In the upper portion of bedrock, contiguous strata were usually found to be separated by thin layers or seams of residual clay. At greater depths, however, the strata were usually in close contact, although separated by a distinct break. In nearly all cases, these planes were sufficiently open to be water bearing, or potentially water bearing.

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[23] "Geology of Norris Dam Site," by Arthur Keith. Unpublished report to TVA, August 30, 1933, p. 7.

The rock involved in the dam site was entirely free from faults, although it was extensively jointed. Most of the joints were merely incipient structures, along which the rock broke on blasting, but some of them were fissures, commonly somewhat enlarged by solution. Some of the joints thus enlarged were open, but in most cases they were more or less completely filled with residual clay. Joints exhibited a great diversity of trends, but most of them were characterized by northeast trends and steep dips.

As is almost always the case in limestone and dolomite, the rock forming the foundation, abutments, and adjacent portions of the reservoir rim was, to some degree, cavernous. This condition was clearly recognized from the start, and construction was not undertaken until its seriousness had been fully ascertained. With the exception of a cave—6 to 10 ft wide and 10 to 25 ft high—in the right (west) abutment at El. 980, the openings resulting from solution of the rock were found to be of relatively small dimensions, except for lateral extent. Cavities were appreciably larger and somewhat more abundant in the abutments than in the foundation. Drilling revealed that cavities and seams ranging in vertical dimensions from less than 0.1 to 3.0 ft occurred in the foundation to a maximum depth of 86 ft below the original bed of the river. These cavities averaged slightly more than 0.3 ft in vertical thickness, and most of them were filled or partly filled with residual clay. Nearly all of the small cavities encountered in the foundation seemed to represent merely enlarged stratification planes, but the larger ones in the abutments were, for the most part, developed along joints. Some strata were found to be decidedly more cavernous than others because of their greater susceptibility to solution.

The cavities were not sufficiently large, open, or closely spaced in either the abutments or foundation to render the rock incompetent to bear the load to be imparted by the structure. Nevertheless, they constituted serious leakage hazards and made necessary the extensive and systematic treatment with which Mr. Lewis' paper deals. Without such remedial treatment, leakage undoubtedly would have occurred as soon as the reservoir began to fill. As leakage progressed, the clay filling would have been washed out of the cavities and seams, resulting in the loss of increasing volumes of water. The cost and difficulty of stopping, or even appreciably retarding, leakage that has developed in limestones or dolomites under and around a large dam can be thoroughly appreciated only by one who has attempted such a task.

The systematic washing of clay out of seams—most of which were along bedding planes and consequently inclined downstream at an average of 4°—and refilling the openings with grout removed the very slight possibility of failure by sliding within the foundation and the probability of settlement under the load of the dam, as well as leakage hazards.

The advantageous use of large-diameter drill holes in foundation exploration and foundation treatment is justifiably emphasized by Mr. Lewis. Norris is one of the first large dams in the United States at which large-diameter core drilling was done on a moderately large scale for both exploration and foundation treatment.

The paper by Mr. Ross also deals with a dam site located on limestone. Although the Guntersville site is quite different from both the Chickamauga and Norris sites, it has some geologic features in common with each of them. On the one hand, it resembles Norris in the simplicity of the geologic structure and in the relatively small amount of deep cavitation. On the other hand, it resembles Chickamauga in that much of the limestone is quite pure—and consequently quite soluble—and in the presence of a shaly stratum which has effectively controlled the depth to which solution cavities were formed.

During the early stages of the preliminary studies, the Bangor limestone as a foundation rock was looked upon with considerable suspicion. This suspicion was not entirely unfounded, for the largest caves in the entire TVA area are in the Bangor limestone. Moreover, the cavernous character of the rock is well attested by the enormous leakage under the only dam in the area to be founded upon the formation. It was not until seven other sites had been found unsuitable that the Guntersville site, previously known as the Coles Bend Bar site, was tentatively selected. A considerable amount of core drilling revealed that the site as a whole was unusually good.

As Mr. Ross clearly points out, the only serious foundation problems were those involving solution cavities, nearly all of which were above the shaly stratum. The most serious problem was occasioned by deep and extensive zones of solution under the south (left) earth embankment. Remedial treatment of the south earth embankment area was accomplished mainly by excavation to good rock and backfilling with impervious material and by grouting. The general condition of the upper portion of bedrock—which prevailed over much of the dam site—is well illustrated in a block diagram (Fig. 27) and a photograph (Fig. 28).

R. F. WALTER,[24] M. AM. SOC. C. E. (by letter).—The interesting paper by Mr. Lewis comes at a time when foundation treatment is steadily becoming a more important factor of dam construction due to the decreasing availability of ideal sites. The constant desire for additional safety makes careful foundation exploration and thorough treatment a prerequisite of modern dam building. The author's account of the work at Norris Dam, together with his description of methods used in obtaining the information necessary for adequate treatment, makes this paper of value to every engineer connected with foundation work.

No two dam foundations are alike but many may have characteristics similar to others. Solution channels through limestone were also characteristic of the formation underlying the immediate foundation of Alcova Dam in Wyoming. This dam, an earth-fill structure with a maximum height of 265 ft, a length of 763 ft, and a total volume of 1,600,000 cu yd, was placed on a foundation consisting of two formations dipping about 15° downstream. Thin-bedded, limey sandstone composes the abutments and immediate foundation for the dam, but this in turn is underlaid by similarly tilted thin-bedded limestone containing numerous solution channels. With the exception of one cavern under the upstream toe of the dam, which was found to contain fine sand, the solution channels were entirely clean, most of them containing warm water under a pressure of about 20 lb per sq in. as measured at original river level, and therefore required no washing as did those at Norris Dam. Warm water at the rate of approximately 10 cu ft per sec was discharging into the foundation area.

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[24] Chf. Engr., Bureau of Reclamation, Dept. of the Interior, Denver, Colo. Died June 30, 1940.

The problems at Alcova were (1) to intercept and stop the inflow of water into the foundation area under the impervious section of the dam; (2) to consolidate this same foundation area; and (3) to provide impermeable cutoffs under the dam to prevent loss of reservoir water. All of these problems were solved successfully by pressure grouting. Only the grouting operations required to stop the inflow of water into the foundation area will be discussed.

Similarly to the reservoir rim tightening at Norris Dam, it became evident almost immediately that considerable quantities of grout would be required to fill the numerous solution channels, and it seemed advisable to use a grout containing sand. Fortunately, there were deposits of a fine dune or blow sand just downstream from the dam site which only required screening through a common window screen (to remove an occasional rock) to obtain a sand that, when added to the grout, could be pumped through the regular piston-type pumps commonly used in pressure-grouting operations. Thereafter, as soon as it was established that a hole was connected to a solution channel, a grout consisting of 1 part cement to 2 parts fine sand and 2 to 3 parts water (by volume) was used. It is to be noted that a grout containing sand was used only in those holes that took grout freely and, in nearly all cases, in considerable quantities. An attempt was made to use the sand mixture in grouting the seams and joints in the sandstone but this was quickly abandoned as it was found that there was a tendency for the holes to plug before the seams were thoroughly filled.

The nature of the formation made it imperative that all the grout possible be injected into each hole as a parallel hole 5 ft distant would often miss the irregular pattern of solution channels. Thus there was no way of beginning to grout at each end of a solution channel and closing in toward the center as was done on the curtain grouting at Norris Dam. Individual holes were stage-drilled and grouted to completion before another hole within 40 ft was allowed to be drilled, except in cases where there were two or more grouted holes intervening. The major volume of grout placed in the solution channels was injected through 10 holes which took a total of nearly 140,000 cu ft of the sand mixture as measured prior to mixing. The largest quantity injected into one hole was 38,021 cu ft. No concern was felt over the possibility of grout traveling into regions outside the dam area as it was realized that the only way of being positive that all openings were completely filled was to continue injection to refusal. Furthermore, any grouting unavoidably accomplished outside of the immediate foundation would tend to augment the seal and, therefore, would be not entirely without value.

As grouting progressed, cores were recovered containing grout varying from a thin film to a maximum thickness of 27 in. These cores indicated that a good dense grout had been deposited and showed only very slight evidence of the fine sand having segregated from the mixture. In cases of large openings the first grout placed was found to have shrunk somewhat, leaving a small passage between the top of the grout and the overlying rock. These passages were grouted with a neat cement grout using a water-cement ratio (by volume) of from 3 : 1 to 1 : 1. As the work neared completion, many grout cores were recovered having good bond with the overlying as well as with the underlying rock. The best criterion for ascertaining the effectiveness of the grouting was the reduction of water flowing into the excavated foundation area. This was finally reduced from an estimated flow of 10 cu ft per sec to less than 15 gal per min.

WILLIAM F. PROUTY,[25] ESQ. (by letter).—The paper by Mr. Fox concerning the foundation of Chickamauga Dam, in Tennessee, demonstrates the benefit of close harmony by the engineers and the geologists, in dam construction, and the great importance of the accurate determination of geological formations and structure before foundation excavation is begun. It also demonstrates the great importance of continued study by the geologist during the entire period of construction. The present is an era in which engineering projects involve such great expenditures and such far reaching public liabilities that it becomes necessary to have every possible check on both permanence and safety.

At Chickamauga the close relation of major unsoundness to structure and formational character is revealed strikingly. The excellent cross sections, showing in great detail the thickness, character, and structural weakness lines in the foundation, made possible accurate planning in regard to methods and costs of excavation and foundation improvement. Both the engineers and the geologists are to be congratulated on solving the foundation problems at the Chickamauga dam site.

The detailed geological studies at this locality, based on many deep borings, have brought out or emphasized a number of interesting facts: The bentonite beds are thicker, more numerous, and occur through a wider geological range than was previously thought; the bentonite beds also have great variability in regard to physical properties and permeability.

The so-called "key shale" or bentonite beds at the top of the lower Trenton limestone formation has limited the downward solution in the upper Trenton limestone throughout the area of its occurrence. The deep solution areas are associated with, and chiefly overlie, the zones of shear or intense strain. These lines of unsoundness in the Chickamauga area result from intense northwest-southeast compressional forces. The major overthrust from the southeast has its outcrop a short distance to the southeast of the south abutment of the dam. The numerous secondary faults of comparatively small heave and throw are largely parallel to the major fault and, like it, tend to have less angle of inclination in the near surface zone. It is interesting to note also that in the lower Trenton and upper Lowville, where the bentonite beds are developed, numerous minor faults are to be found frequently with steeper angles of inclination than the main faults and extend only between the beds. This has been caused, apparently, by the greater ease of thrusting along the weak bentonite beds. This type of faulting is particularly well developed where the more resistant member lying between the bentonite beds thickens by lensing in the direction of the thrust. Another type of faulting illustrated in the cross sections is that caused by local unsymmetrical folding, resulting in a greater thrust of the overlying beds in front of (northwest of) the fold.

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[25] Head, Dept. of Geology, Univ. of North Carolina, Chapel Hill, N. C.

The deep solution in the limestone in the area of Chickamauga Dam, as revealed by the core drill, bears out observations made in other limestone areas that solution is not limited, as thought by many, largely to the level of the present water table. There must be some deep-seated artesian flow in the area, or the area was once more highly elevated and the water table relatively lower than now. Although the latter explanation is a possible one, it is less probable than the first explanation.

The geological records from such extensive drillings are of value not only to the engineer in connection with the construction of the Chickamauga Dam, but the generalizations to be drawn from the numerous detailed cross sections produced are of such value as to be of use to the engineer in other areas with somewhat similar rock and structural conditions.

Jacob Feld,[28] M. Am. Soc. C. E. (by letter).—The Symposium papers give a complete description of the investigation, plan of treatment, and cure of the rock rebuilding under the three dams in the Tennessee Valley. The reports will become the standard reference for any future problems of similar nature; the great detail in which the various schemes are given will eliminate many costly researches and experimental developments for such future problems. The idea of strengthening rock for foundation purposes, as well as for the reduction of porosity, on such large scales, is revolutionary. In previous years, the discovery of such porous rock conditions would have meant the abandonment of the site.

It would aid the reader in establishing his perspective if the total cost of exploratory work and grouting were given for each dam, together with the corresponding total cost.

The solution channels were apparently sub-bed flow channels. What changes in the ground-water levels and in the ground itself are expected downstream of the dams, where a complete stoppage of such subterranean flow has been provided?

Mr. Lewis' discussion of the economy of inert admixtures to the grout (see heading, "Use of Rock Flour"), pointing out that such admixtures retard the time of set and therefore that greater areas are treated from any one hole, might be used as an argument for the use of such admixtures with an increase in hole spacing. If the admixtures are finer than cement, especially if they are of colloidal clay (such as bentonite), the grout will travel through finer crevices and will seal such crevices not only by the setting of the cement but also by the swelling of the colloidal gel. Such admixtures also eliminate the formation of fine shrinkage seepage channels in the cement grout, through which water flow will continue and tend to dissolve the limestone, as well as the cement. Water in fine channels gives greater surface contact with the limestone per unit volume and thereby speeds the rate of chemical solution.

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[28] Cons. Engr., New York, N. Y.

The depletion of ground water from the subsurface below each dam during low-water periods will be followed by rapid filling of the voids after heavy rains. The surfaces in the cavities have meanwhile been exposed to drying and oxidation. Has any provision been made for keeping the ground-water table sufficiently high below each dam to eliminate such alternating drying and wetting, which would accelerate future increase in ground cavities?

A. WARREN SIMONDS,[27] ASSOC. M. AM. SOC. C. E. (by letter).—A valuable and instructive treatise on the art of foundation grouting of dams is contained in the paper by Mr. Lewis. The proper foundation treatment is one of the major items of importance in the construction of modern dams. If this work is done properly at the time the dam is built, it will minimize the expense and effort of regrouting the foundation at some later date after the reservoir has filled. Moreover, regrouting the foundation beneath a dam under water load may cause undesirable alteration of the distribution of stress at the base of the dam as a result of changing the distribution of uplift pressure. Therefore, an effective job of foundation grouting at the time of construction is desirable.

The problems connected with foundation grouting vary widely; at one dam site they may be radically different from those at some other site. This variation makes it necessary to alter the grouting procedure to fit the field conditions as the occasion demands. Occasionally it is necessary to vary the grouting procedure at adjacent holes in the same formation.

Mr. Lewis states that the foundation at Norris Dam was composed of a hard banded dolomite with numerous seams, some of which were partly filled with clay and some partly open. In grouting seams partly filled with clay, the seams should be well washed before injecting cement grout. If cement grout becomes mixed with clay, the resulting mass may be slow in setting and it is doubtful if such a mixture will develop much strength.

The patterns of wash and grout holes at Norris Dam thoroughly covered the foundation. The exploration of the foundation seams presented numerous problems which were ingeniously solved by the use of mechanical feelers or "hole explorers" and periscopes. The use of expanders or packers in washing the seams would insure the injection of wash water and air into the seam. In washing seams the question often arises as to the thoroughness of the washing operations. There is a possibility that the air and water may cut channels through the clay into the adjacent holes without completely washing the seam.

It is noted that upheaval gages were used to detect small movements in the upper bedrock so that careful control could be made of the grouting pressure. The ideal pressure used in injecting cement grout in foundation rock is the maximum that can be sustained without causing lifting or displacement of the foundation rock. This results in a wider dispersion of the grout, thereby permitting a maximum benefit to be obtained from each hole drilled. Since there are no set rules for the establishment of a safe maximum pressure, the use of mechanical devices for the purpose of detecting small movements of the foundation is beneficial in regulating the grouting pressure. Surveys for the control of uplift movement have been made with precise levels, Whittimore type strain gages, extensometers, tiltmeters and, if the dam is under load, with uplift pressure pipes. The extent of the observations for uplift depends on the type of rock in the foundation and on the external load applied.

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[27] Engr., U. S. Bureau of Reclamation, Denver, Colo.

The grouting equipment described by Mr. Lewis is of the general type now in use at most grouting jobs. The air-driven or steam-driven duplex reciprocating pump is the most satisfactory machine for placing cement grout. The 7 by 5 by 10-in. size used at Norris Dam is not as flexible as the 10 by 3 by 10-in. size and may not be as satisfactory except in grouting large cavities at low pressure. In place of cast-iron fluid pistons with rubber packing, the writer believes that a medium-hard rubber piston on a steel core is more satisfactory. These pistons are slightly flared at the ends so as to produce a wiping action in the cylinder. The liquid end cylinders of the grout pump should be equipped with liners having hardened steel sleeves. Mr. Lewis states ("Shallow Grouting: Equipment") that "Piston packing and valves were usually replaced in the field, although it was necessary to have cylinder liners removed and replaced in the machine shop." Pumps are now being constructed in which the liners can be removed and replaced in the field, thereby eliminating the necessity of machine shop service. The writer agrees with Mr. Lewis in his statement, "Valve disks of medium rubber were found to be far superior to the fiber disks furnished as original equipment." A more satisfactory arrangement of the equipment used at Norris Dam would have included a mechanically agitated sump between the mixer and the agitator as Mr. Lewis suggests.

The water-cement ratio of grout used in foundation grouting is dependent on the type of rock, presence of seams, cavities, zones of broken rock, etc. The purpose and location of the grout holes are also factors in determining the most suitable grout mixture. The most successful grout mixture is the thickest that can be injected without plugging the hole.

An excellent description of the use of packers in grouting operations is given by Mr. Lewis. There is considerable variance of opinion among engineers as to the relative merits of packer grouting and stage grouting. The procedure in grouting the foundation at Grand Coulee Dam combines the experience of the U. S. Bureau of Reclamation in grouting at other Bureau dams. Grout holes are drilled into the granite foundation until an open seam is encountered. This is known by the loss of the drill water or by a flow of seepage water into the hole. The hole is then grouted progressively in stages; that is, the drilled part of the hole is grouted, the grout is cleaned out of the hole after the cement has taken its initial set, and the next stage is drilling and grouting. A fish-tail bit is used in cleaning out the hole. If the drilled hole is tight, stage grouting with a packer is resorted to and the hole is grouted starting at the lower part of the hole and working progressively upward. Where the foundation rock is fairly tight packer grouting is probably more effective than stage grouting. Where several holes are interconnected by a common seam the use of packers may not be as effective as grouting in stages. Moreover, if a packer is placed in broken or caving rock there is a possibility of grout leaking by the packer and the packer becoming grouted in the hole.

The problems arising in foundation grouting vary widely. At times the engineer in charge may find his ingenuity taxed to the limit in order to effect a successful job of grouting. Mr. Lewis shows that he has given considerable thought and study to the problems arising in treating the uncertain character of the foundation of Norris Dam. The results obtained are worthy of study by all engineers engaged in foundation work.

The paper by Mr. Hays adds needed information on the treatment of the foundations of dams as such material now available to the engineering profession is quite limited. It contains an excellent description of the grouting of a difficult foundation for two types of structures: Concrete structures, which include the lock, spillway, and power house; and the earth structures which include the two embankment dams at each abutment. The paper also includes a description of some experimental grouting under the embankment sections and a description of the temporary construction grouting.

It is interesting to note that the purpose of the experimental grouting under the embankment sections was to determine the most suitable method for making the cutoff under the earth dams. Grouting a foundation which contains numerous small defects under a cover of 40 ft of overburden presents numerous problems and questions to the engineer. One of the most important questions relates to the problem of under-grouting or over-grouting. If the injection of grout is terminated too soon, an ineffective job of grouting will result. If the grouting is excessive, cement will be wasted and the costs of grouting will be increased unnecessarily. Successful grouting of test areas, as was done at Chickamauga Dam, gives confidence to the engineer in charge of the work.

The regular program of permanent grouting under the concrete structures, as described by Mr. Hays, is of the type now generally used in the construction of most dams built by the U. S. Bureau of Reclamation. First, a general consolidation of the surface rock is obtained by blanket grouting through shallow holes. This is followed by grouting through holes of intermediate depth, and finally the deep holes forming the cutoff are grouted. An interesting feature described by Mr. Hays is the inclination of two sets of deep holes in opposite directions in the plane of the cutoff. This procedure is effective in intercepting vertical as well as dipping seams.

The grouting at Chickamauga Dam shows the results of careful planning and execution. The thoroughness of the work is indicated by the re-drilling and grouting of the consolidation system of holes as many as five times in certain areas. The final spacing of drilled, washed, and grouted holes of 12 in. on centers should have reduced the possibility of missing small cavities and solution channels to a minimum.

The program of grouting under the earth-dam sections at the abutments presented numerous complications due to seepage water and a badly distorted geological structure. It is interesting to note that asphalt grouting was tried (and was not successful) in an attempt to grout an underground channel beneath the south earth dam. The development of low-cost grout composed of a mix containing 4 parts sand, 1 part bentonite, and 2 to 3 parts of cement is similar to the mix used by the contractor in grouting the leak in the cofferdam at Grand Coulee Dam where sawdust and shavings were also added.

The use of bentonite combined with cement as an ingredient of a grout mix has been used successfully in temporarily shutting off flows of water. However, it is questionable whether a bentonite-cement combination makes a permanently stable mix. The same question may arise in the case of the clay-cement combination when subjected to repeated soaking and drying.

It is noted in the discussion of the temporary construction grouting of the first stage cofferdam that 36,350 cu ft of asphalt and pitch were used. It is regretted that Mr. Hays did not describe the method used in injecting this material and the results obtained.

V. L. Minear,[28] M. Am. Soc. C. E. (by letter).—The Symposium on Foundation Experiences of the Tennessee Valley Authority is a noteworthy contribution to the literature on foundation treatment. The different papers not only describe in detail the successful methods used in treating the difficult dam foundations built upon, but, what is of equal importance, they open up for discussion various bothersome questions on foundation treatment in general, which it is believed could be considered with profit.

Among the debatable questions brought up, particularly in Mr. Lewis' excellent paper, are the following:

(*a*) What pressures shall be used in introducing the grout?
(*b*) What sizes and spacings of grout holes shall be used?
(*c*) How much and what kind of grout shall be injected?
(*d*) What methods shall be used?

The writer submits herewith his views on the foregoing questions in so far as they apply to Mr. Lewis' paper. These views, gained from experience in treating the foundations of twelve different dams, are offered with no thought of establishing authoritative rules; rather it is hoped to learn from the experience of others by stimulating discussion.

Pressures.—What pressures shall be used? This is a matter of prime importance as it influences the optimum spacing of holes and the water-cement ratio of the grout to be injected. The highest pressure that can be used safely is desirable, but, unfortunately, there is no safe rule for its determination. For lack of a better criterion some have used an arbitrary value of one or more pounds of pressure per foot depth of hole. This rule is of doubtful value except in special cases. A more rational rule would be to require the application of a specified minimum pressure to all holes. This should be based upon the hydrostatic pressure to which the finished structure will be subjected. To this irreducible minimum, as much additional pressure should be applied as can be used safely. Not only does safe pressure vary with different formations, but it varies greatly from hole to hole in the same formation.

It has been observed that rock displacement does not ordinarily occur on those holes which take initial grout slowly and at high pressure. It is more often the one which consumes it freely for a considerable time that terminates in a bad grout leak indicating rock displacement. This occurs in many cases before the specified maximum pressure has been applied. Consequently, if damage is to be avoided and at the same time effective grouting done, some method other than the application of a uniform maximum pressure to all holes must be used. The use of "upheaval gages" described by Mr. Lewis is a step in the right direction. Some such instrument, more sensitive than the ordinary engineer's level, is fully as essential in foundation grouting as are the dial gages or extensometers used almost universally across the contraction joints of masonry dams when they are grouted.

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[28] Associate Engr., The Panama Canal, Diablo Heights, Canal Zone.

A valuable adjunct to rock grouting, intended to serve the same purpose as upheaval gages, is the portable tiltmeter devised by R. S. Lieurance. This instrument consists of an optical lever containing a pool of mercury which is capable of indicating extremely small movements due to tilting. When such movements occur, the pressure of injection can be reduced accordingly before damage is done. The instrument is light, sturdy, readily transported, and requires no special knowledge for its operation. Its principal advantage lies in the fact that it obviates the necessity for drilling holes and grouting in rods as is required when upheaval gages are used.

There are two criteria in common use for the final application of pressure to foundation holes. The first requires that pumping continue until the hole refuses to take 1 cu ft of grout in some specified time (such as 10 min) at the specified pressure. The second criterion is that injection shall continue to refusal at a pressure which is a fractional part (such as two thirds) of the allowable pumping pressure. The writer prefers the latter method. He and other engineers experienced in this line of work believe that requiring the contractor to pump on each and every hole until that hole consumes less than 1 cu ft of grout in, say, 10 min of pumping, works a hardship on the contractor with but little benefit to the owner. They argue that it is as necessary to keep grout agitated in the pump and delivery line as it is in the sump if cement is to be prevented from settling out of the grout and that when grout is being introduced at so low a rate, the velocity is so low that most of the cement settles out of the grout stream and finds lodgment in the pump and delivery line while the injectamenta is largely water. It is thought that, so far as the quality of the work is concerned, the foregoing are matters of little moment; that the hole has been grouted, successfully or otherwise, long before the final pressure is applied; and that the selection of a proper water-cement ratio, at the proper time, is of vastly more importance than the injection of the ultimate cubic foot of liquid into what is probably a plugged hole.

Hole Layout.—What size and with what spacing shall grout holes be drilled? This is a question of fundamental importance since the drilling of holes constitutes a major item in any rock grouting program. The cost of such drilling is ordinarily influenced by the diameter and depth of the holes. This is particularly true when the condition is such that diamond-set bits must be used. The unit prices in Table 4, which were bid on a large western dam, are typical of this tendency.

Under similar conditions, for any given sum of money that can be expended in drilling, more holes and consequently a closer spacing can be used when small-diameter holes are drilled. Since the goal of rock grouting is to fill the rock fissures, and since the pattern which these fissures follow is unknown, it would seem that the closer the spacing of holes, the greater the probability that all seams will be filled and hence that that diameter of hole is most desirable which can be drilled most cheaply, provided that the cheap hole allows grout to be introduced as effectively as does the more costly hole.

TABLE 4.—UNIT BID PRICES, IN DOLLARS, FOR GROUT AND DRAINAGE HOLE DRILLING

Diameter of holes (inches)	DEPTHS IN FEET					
	0 to 30	30 to 50	50 to 75	75 to 100	100 to 150	150 to 200
$1\frac{3}{8}$	1.40	1.70	1.85	1.90	2.00	2.10
3	2.50	3.00	3.00	3.25	3.50	3.50
$5\frac{1}{2}$	5.75	5.75	....	....	....	....

The quantity of grout that must pass each unit area of the hole's cylindrical surface is greater in small holes than in large holes, and therefore it might be expected that small holes would tend to become obstructed. This is true. The writer has noted results similar to those mentioned by Mr. Lewis under "Curtain Grouting: Action of Grout." However, this characteristic does not justify the drilling of oversize holes, especially in the light of experience. Data on the record hole on each of three large dams constructed during the decade 1930–1940 are as follows:

Dam	Hole diameter (inches)	Solids injected (cubic feet)
A	1	5,000
B	$1\frac{7}{16}$	28,800
C	$1\frac{1}{2}$	38,000

If 28,800 sacks of cement can be introduced through a single $1\frac{7}{16}$-in. hole, the necessity for drilling larger holes is not apparent. It would seem that the quantity of grout that can be injected depends more upon the structure penetrated than it does upon the diameter of the hole drilled. Hence the writer is of the opinion that, other conditions being equal, that design is most desirable which provides the maximum number of closest spacing of holes, regardless of diameter.

Quantities.—How much and what kind of grout shall be injected? Cement usually constitutes the second most costly item in a grouting program. The writer is in complete agreement with Mr. Lewis' conclusions that:

(*a*) "In the future, the major economies that will be effected in the field of foundation treatment will probably result from the development of cheaper materials for grouting" (see heading "Conclusion").

(*b*) "* * * it [is] unwise to use sand for the primary grouting of seams in a foundation that will be subjected to more than a moderate head" (see heading "Use of Rock Flour: Sand for Grouting").

Among the emergencies which arise and must be coped with by the field forces without the delays incidental to consultations and special studies, as suggested by Mr. Lewis under the heading "Conclusion," is the case of the "unusual" hole that consumes an exorbitant amount of grout. It is true that the quantity of grout that can be injected into a hole is very materially reduced by using a sanded mix or by pumping thick grout slowly, but such procedure is thought to be a form of self-deception. The grout consumed is "exorbitant" because a weakness has been disclosed by the grouting which exploratory drilling failed to expose. Using a sanded mix or pumping thick grout slowly merely masks what may be a highly undesirable condition. Filling the reservoir can well be associated with excessive hydrostatic uplift pressures and leakage through the dam abutments. Regrouting then becomes a necessity. The writer has had occasion to so regrout three major structures and hence looks with extreme suspicion upon attempted economies in the quantity of grout that a hole will consume.

Methods.—It should be borne in mind that the job will be made safe in the field and not in the office. There is perhaps no phase of engineering or construction which does not lend itself more readily to rule-of-thumb or handbook methods than does foundation grouting. Exhaustive preliminary studies, competent design, stringent specifications, and excellent equipment lose much of their value when the actual work is done by low-grade labor under mediocre supervision. The field methods used are of vital importance. How much dependence can be placed in so-called washing operations? Should screened cement be required? Should stage grouting be used? Space does not permit giving much time to these important questions. It is believed that seam washing should be done as thoroughly as the author describes, or it should not be attempted. The writer has had occasion to expose, by excavation, seams which had been incompletely washed before grouting and found the contained grout to be a putty-like paste, the clay having intermingled with the grout to an extent which prevented the cement from taking a set.

Should rescreened cement be used? Fineness is undoubtedly desirable in grouting the fine seams in foundation rock. However, rescreening costs are relatively high, and it is possible to remove oversized particles by using the grout delivery line as a wet classifier. This method takes advantage of the sorting action of flowing water carrying suspended matter. No deposition takes place as long as a hole consumes grout freely, but as it tightens up the velocity of the flowing grout decreases, thereby depositing the oversize particles in the invert of the pipe line from where they are readily removed by occasionally sluicing out the line.

Should stage grouting be used? Although this is a more costly method than full-depth grouting, the additional cost is thought to be warranted in many cases. The openness of seams in rock ordinarily decreases with depth, and consequently the grout consumed per foot of hole is greater near the surface where shrinkage is greatest. Grouting in sections from the bottom of the hole upward, by means of packers, permits the use of higher pressures in the lower reaches and eliminates the necessity for cleaning the hole after each grouting. However, it does not subject the near-surface rock to the repeated groutings of the so-called "stage" method. It is the successive introduction of grout into this critical zone which compensates for shrinkage and holds to a minimum the tendency for grout to return from "holes that had been previously grouted to refusal" (see heading, "Shallow Grouting").

C. E. Blee,[29] M. Am. Soc. C. E. (by letter).—All of the papers presented in this Symposium are full of interest in that they give the geological conditions encountered and the foundation treatment developed to meet these conditions. The writer has been particularly interested in the foundation treatment at Chickamauga Dam, having had the opportunity of visiting this work from time to time while it was in progress.

The grouting used as a construction measure to tighten the cofferdams and to reduce the flow of water into the cutoff trench excavation might be regarded as of secondary importance, but actually was a major operation and quite vital to the completion of the entire project. The development of what Mr. Hays has termed "prescription" grout, consisting of sand, bentonite, and cement, is of particular interest. Where the quantities involved reach such large amounts as they did at Chickamauga Dam (with one item of "prescription" grout exceeding 150,000 cu ft), it seems almost an economic necessity to develop a low-cost grouting material. Along the same line is the development of the stabilized clay grout, formed by the addition of portland cement to a local clay, used to fill cavities outside of the cutoff but under the earth embankment

The grouting under the concrete portions of the dam was more along conventional lines. It consisted of consolidation grouting and cutoff grouting similar to that which might be encountered in any concrete dam but more extensive than is usually found.

In reviewing the procedure used for obtaining the cutoff in the earth-dam sections, one is impressed by the thoroughness, yet flexibility, of the methods used. The 36-in. core drill, although introduced primarily as a means of exploration, has developed into a very useful construction device for cutting a shaft to give access to underground work. At Hiwassee Dam, in one particular area of the foundation, there was a comparatively narrow seam of disintegrated rock overlaid by about 20 ft of sound rock. In order to treat this condition without removing the overlying sound rock, a number of 36-in. holes were drilled in this particular area. Workmen were then able to go in and, operating from these holes by mining methods, successfully cleaned out the seam of disintegrated rock, which was then backfilled with concrete and later pressure grouted.

After reading the paper by Mr. Hays, it appears that there should be no question as to the adequacy of the cutoff obtained in the earth-dam sections. Rather, there might be a question as to whether a less extensive treatment would have sufficed. With respect to engineering structures in general, and dams in particular, this is a question that is seldom answered even by experience with the structure in use. In the case in question, it is evident that steel sheet piling could not be used on account of the mass of boulders and rock fragments overlying bedrock. The open trench excavation was the logical method of reaching bedrock and had the further advantage of affording a means of excavating through the upper and more disintegrated portions of the limestone. Mr. Hays states that such excavation, together with the resulting backfill, was found to be more economical than the excessive grouting that would have been required to fill the openings in this surface rock.

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[29] Project Mgr., Fort Loudoun Project, TVA, Lenoir City, Tenn.

The adopted spacing of 12 in., center to center, for the final grout holes in the cutoff may seem rather on the conservative side. It must be recognized, however, that the cost of these holes, driven with wagon drills, is not a large item and if half the holes had been omitted, no enormous saving would have been realized. In looking at the cutoff on the site, particularly near the outer limits of the south embankment, one was apt to have the impression that with so much overburden a long leakage path would be presented which would materially aid in accomplishing the cutoff. Apparently, however, from the large amount of pumping required during construction, open water passages extended to the outcropping of the rock in the river channel, and the cutoff had to withstand the full hydrostatic head of the reservoir.

The writer agrees that, in constructing a cutoff by grouting, it is advisable to place the holes all in one row with "the pickets of the fence" close together rather than to use staggered rows.

The paper by Mr. Hays describes one of the most extensive projects ever to be undertaken in foundation treatment for a dam. It is particularly valuable in showing the possibility of making available dam sites which might otherwise have been considered impracticable.

BARTON M. JONES,[30] M. AM. SOC. C. E. (by letter).—The subject of Mr. Lewis' excellent paper is of fast growing importance along with improvements in the methods and procedure of treating dam foundations. As time goes on, dams will be constructed upon foundations less perfect than those now available. Practices developed to make the less perfect sites satisfactory at costs that are feasible and to obviate the need for removing large masses of somewhat inferior rock and replacement with costly concrete will be of inestimable value.

The paper applies mostly to the horizontally bedded type of formation. The Norris foundation, of this type, was extremely open and the filling of the seams and cavities was possibly more necessary from the standpoint of bearing than from watertightness, although both were of prime importance. The paper does not refer to the inadequate bearing resistance.

Mr. Lewis is to be commended for bringing together and describing the many and varied operations and pieces of equipment used in the foundation treatment and in mentioning what may seem to be some of the minor details that are often omitted in papers but which in reality supply extremely useful information to those having such work in hand. Also, he should be commended for the frank occasional admission that some other method or device, if it had been used, would have been better.

The effectiveness and moderate cost of the treatment of the Norris foundation reflect the continued care and study given to all situations as they arose and in formulating plans that would be most suitable in each case. The dam is practically bottle-tight. With reference to tightness against leakage, it should be explained that evidence indicates that the largest item of water from drain holes mentioned in the paper, namely 0.55 cu ft per sec from under the spillway apron (see heading "Curtain Grouting: Leakage"), is supplied mostly from sources in the sidehills and abutments rather than from the reservoir.

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[30] Cons. Design Engr., TVA, Knoxville, Tenn.

The paper does not explain that the adopted system of patterns and sequence used in drilling the grout holes was effective as well from the standpoint of drilling progress and economy as for both the washing and grouting operations.

The parallel tunneling on one large seam in the east or left abutment might warrant some amplification beyond the description given by Mr. Lewis to reveal other important features. To avoid weakening the overlying slab, it was left intact. It formed the roofs of the several parallel tunnels which were excavated entirely downward from the seam, in the under-side layer of rock. The 20-ft parallel spacing of the tunnels allowed a thorough cleaning out and washing of the seam between tunnels under dry accessible conditions and refilling with dry packed cement mortar where the opening was sufficient to permit it. Very satisfactory and rapid concreting of the tunnels was done through 36-in. holes at their far ends, drilled from the surface, and through which concrete was dumped in 3-yd batches with considerable impact. Heavy bulkhead forms were built to retain the concrete when it reached the portals.

It may be noted that all grouting was done at relatively low pressures considering the height of the dam. The low pressures proved entirely adequate. The aim was to use pressures as high as possible, but limitations were brought about by measurable effects detected by upheaval gages installed for measuring vertical movement of the rock at its surface, as explained in the paper. Under the completed dam a pressure of 150 lb per sq in. was frequently found to be too high. The upheaval gages (heading "Curtain Grouting: Upheaval Gages") were first proposed by the writer and perfected by the engineering force at Norris Dam. A subsequent design supplied by the Bureau of Reclamation was not used because dial indicators made the device too delicate for use around heavy construction operations.

It is believed that core drills making holes large enough for a man to enter and inspect the undisturbed formation—such as the 36-in. shot drills—were used first at Norris Dam where the immediate result was a reduction in foundation stripping with important savings in construction cost. It might be suggested that grouting a single small hole in the center of a proposed large hole, instead of four holes outside, would ordinarily close the surrounding water-bearing seams sufficiently to permit nearly dry drilling.

The paper by Mr. Lewis might have directed more attention to the uncertainty of indications as to the amount of seams and cavities as shown by cores from holes of small drills, such as $2\frac{1}{16}$ in. for example. Also, it might have emphasized the variation of indications due to causes such as slow and steady drilling versus careless drilling, the use of a double versus single barrel, and the frequent pulling of cores. To the use of the double barrel, Mr. Lewis ascribes a core-loss indication of 25% reduced to 6% in the same rock. Small holes generally, or invariably, exaggerate the defects of the formation drilled.

The rock flour, mentioned by Mr. Lewis as an admixture to the grout used in the rim treatment, was a by-product. The dolomite sand, produced by hammer mills, was washed and the wash water passed through an improvised classifier which divided the suspended particles at a size of about 200-mesh. The material coarser than 200-mesh was used to balance the grading of the sand used for concrete. Material smaller than 200-mesh was stored for future use as fertilizer, and it is this material (called "rock flour" by Mr. Lewis) that was used in some of the grout.

JAMES S. LEWIS, JR.,[31] ASSOC. M. AM. SOC. C. E. (by letter).—The writer would like to express his appreciation to those who contributed their opinions and criticisms of the material presented relative to the foundation experiences of the TVA and to acknowledge the great value of the information and ideas contained in the discussions of the Symposium. It is probably worth remarking that the authors of the Symposium papers, and the discussers, seem to be in general agreement upon certain features that could well be re-emphasized. For instance, the inadequacy of small core borings for obtaining comprehensive information about foundation conditions is recognized and the use of large core holes, which permit visual inspection of the undisturbed foundation, is becoming more common for exploration purposes. The economy of cofferdam grouting with cheap grout before unwatering, if the need is indicated, will probably not be questioned by one who has experienced the excessive operating and pumping costs attendant upon the maintenance of a wet cofferdam. The discouragingly difficult and expensive task of stopping foundation leaks, which may appear after water has been impounded, justifies the use of every reasonable precaution to prevent them.

Mr. Moneymaker's lucid description of the geology of the various foundations described in the Symposium adds materially to the information presented by the authors in describing the problems peculiar to the different sites. There is a growing inclination on the part of engineers to take greater advantage of geologic skill in connection with the construction of large dams, and the value of having a resident geologist attached to a project throughout the duration of construction operations is recognized in the TVA. The opinions of the geologists have proved especially valuable when questions involving the necessity of removing rock of dubious quality have arisen, and sizable economies have resulted from the immediate availability of expert geologic opinion. It seems unnecessary to state that, ordinarily, every yard of rock excavation avoided is also a yard of concrete saved.

As Mr. Walter declares, no two dam foundations are alike, but the same fundamentals of successful treatment are frequently found to be applicable to widely separated sites. Although the methods used at Alcova Dam and at Norris Dam differed as to details, the problems encountered and the solutions found were similar in many respects. The successful use of fine dune sand as a medium of reducing costs when large openings were to be filled is another example of the manner by which economies may be achieved with materials available at comparatively little cost. The use of this material also doubtless reduced shrinkage as compared with neat cement grout and, as a result, formed a superior seal.

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[31] Constr. Supt., Watts Bar Dam and Steam Plant, TVA, Watts Bar Dam, Tenn.

The wisdom of increasing the spacing of holes, when admixtures that retard the setting time of the grout are used, is questionable. Narrow channels, such as result from the weathering of vertical joints, may well be missed by the drill holes if they are not spaced at reasonably short intervals, and the writer feels it unwise to increase the uncertainties necessarily attendant upon work of this nature. The following costs, which were incurred at Norris Dam, are included at the suggestion of Mr. Feld:

Drilling, foundations	$ 234,286.75
Drilling, reservoir rim	138,018.13
Grouting, foundations	309,235.10
Grouting, reservoir rim	138,091.27
Excavating and filling tunnels	124,013.34
Total cost of all treatment	$ 943,644.59
Total direct construction cost of Norris Dam	$18,584,745.00

As Mr. Simonds states, it is essential that clay-filled seams be washed free of all loose material before grout is injected into them. In tunneling to seal an 18-in. clay-filled seam, a section of which had been grouted, under the east abutment of Norris Dam, an excellent opportunity was afforded to observe the action of grout in clay. This seam had not been washed, and excavation revealed that the grout had worked its way into the clay in fins and layers, filling any openings that might have existed and definitely consolidating and increasing the density of the material generally. However, despite the noticeable improvement, the seal was not positive and the condition was not such as to resist percolation safely and to resist possible subsequent erosion for an indefinite time. As Mr. Simonds indicates, engineers are frequently at variance in choosing between stage grouting and packer grouting. Both methods possess advantages as well as disadvantages. When time is limited, the use of packers tends to expedite the operations. Also, the expense of cleaning out or re-drilling holes between stages is eliminated. Stage grouting, as a result of the repetitive process, should tend to form a tighter seal by filling any contraction spaces that may exist around grout of previous injections.

The questions raised by Mr. Minear in his interesting discussion might profitably be asked in advance of every program of foundation treatment—and the answers would probably be different for every site. These questions deal with the fundamental problems common to the grouting of all types of foundations, and the correct answers supply the key to satisfactory results and economical costs.

The determination of the proper pressure of injection usually depends somewhat upon experimentation, and it is generally safe to say that a horizontally stratified rock should be given all that it will stand without serious upheaval. The tiltmeter, which Mr. Minear describes, would certainly have decided advantages over the comparatively expensive installation of upheaval gages used at Norris Dam. However, the writer questions whether it would not be possible for simple vertical movement to take place without registering on the tiltmeter. Angular displacement would occur inevitably if movement continued, but it seems possible that damage might result before the tiltmeter registered.

The writer feels that fairly small holes may be used successfully for grouting when it is unnecessary to wash loose material from underlying seams. Holes of large diameter make it possible to insert devices for washing individual seams but, for grouting purposes, they probably possess no advantage over small holes. The higher fluid velocities that would exist in the smaller holes at any given injection rate would tend to keep them scoured clean and to prevent plugging in the hole. Mr. Minear's reasoning regarding the relation between the cost of drilling and the spacing of grout holes is logical and clearly stated.

Concrete-filled tunnels probably form a more positive seal against percolation than any other practical means that might be employed; and where seams are large and extensive, the cost of tunneling, which may be estimated with fair accuracy, may compare favorably with the cost of grouting. As Mr. Jones states, the tunnels at Norris Dam were located so that the seams which were followed formed the roof, and he indicates that this location was chosen in order to avoid disturbing the overlying strata of rock. It is extremely difficult to fill a tunnel to the top, completely, with concrete. It becomes necessary, therefore, to grout the roof of the tunnel after it is filled and quantities of grout may escape laterally to the seam, the desirability of which may be determined by the circumstances. If conditions do not warrant grouting the seam, in addition to sealing it with a tunnel, a better condition will result from locating the tunnel so that the seam forms the invert. When the tunnel is filled, the seam will be sealed completely, and any space remaining between the concrete and the roof of the tunnel may be filled with a relatively small quantity of grout.

The upheaval gage used at Norris Dam was designed by Mr. Jones and served its purpose very effectively.

Robert M. Ross,[32] **Esq.** (by letter).—When the paper was written, the water had not yet been impounded behind Guntersville Dam. Now that the lake has been filled for nearly two years, it may be of interest to review the resulting changes in ground-water conditions, in the flood plains downstream from the embankments, and to note the effectiveness of the cutoffs and grouting in the foundation and of the bulkhead in the nearby cave.

Leakage under the dam and around the abutments has been negligible. The total flow from the well points located in the flood plains, downstream from the embankments, may be considered of no engineering or economic consequence. Nevertheless, the behavior of the ground water in the flood plains is of considerable interest.

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[32] Asst. Geologist, TVA, Water Control Planning Dept., Geologic Div., Paris, Tenn.

Before construction was started at Guntersville Dam, the area at the foot of the north abutment and downstream from it was damp and boggy during a large part of the year and numerous wet weather springs flowed during rainy periods. When the construction plant was built, in 1936, drains were installed and much of the low, boggy land was filled in with earth, rock, and gravel. According to foremen who supervised this work, the fill was sometimes as much as 6 or 8 ft thick. However, even with a system of drains, well point 7, located in this area, 275 ft below the axis, at station 3 plus 10 South, 2 plus 75 West, would flow after heavy or prolonged rains.

In the latter part of January, 1939, after the Guntersville Reservoir was filled, well point 7 began to flow and two small seeps appeared near it, at station 3 plus 8 South, 2 plus 78 West. Several wet, soft places and very small seeps also developed near the northwest corner of the parking lot and about the Time Office. The total flow was between 10 and 15 gal per min, at first. It diminished later, but varied between 2 and 20 gal during February, showing increases after rains. No large increases occurred until the night of March 29 when, after a very heavy rain, a seep delivering about 22 gal per min broke out in the road. It continued to flow until it was diverted through an underground drain. Since March 29, 1939, a flow of between 15 and 30 gal per min has been maintained. In March, a number of additional well points were drilled so that, since then, very detailed information on the ground-water levels has been available. The water flowed from several of these well points until extra sections of casing were added to them. Well point 7-E, located about 2 ft from 7, took most of the water from the latter.

Four methods of determining whether the water coming from the well points and seeps was ground water percolating down from the abutment and the plateau remnant back of it or water leaking from the reservoir were followed. These were (1) a study of ground-water elevations and seepage as related to rainfall and reservoir levels, (2) tracing subsurface water movements by means of fluorescein, (3) chemical analyses, and (4) comparative temperature readings from the water in the well points and in the reservoir.

Lack of space makes it impossible to describe these investigations and their results in detail. It must suffice to say that the ground-water studies, chemical analyses, and temperature readings all indicate unequivocally that the water coming from well points 7 and 7-E and the nearby seeps is ground water. The experiments with fluorescein, having had only negative results, are inconclusive.

Part of the flow from the well points is undoubtedly ground water and, in the writer's opinion, it all is. Nevertheless, before the filling of the reservoir, seepage occurred only intermittently and the water in the abutment stood considerably lower than it has since. The levels in the well points on the abutment only reached an elevation of as much as 585 during rare periods of very heavy rain, usually standing 10 or 15 ft lower, while, since January, 1939, they have never gone below 585. Obviously, a marked change has occurred and the reservoir is responsible. The explanation is probably this: Before the gates of the dam were closed, the ground water in the large plateau remnant against which the dam abuts seeped away to the river underground, passing through a large area both upstream and downstream from the axis. When the reservoir was filled, the outflow area was sharply reduced and the flow of ground water concentrated downstream from the abutment. This upset the previously established equilibrium of flow and produced a considerable rise of the water table in the abutment. The additional hydrostatic head on the outflowing water beneath the foot of the abutment caused it to well up to the surface and issue from seeps and well points.

Mr. Gongwer has given a comprehensive account of the ground-water behavior in the south flood plain and the thorough and detailed investigations that were made there. His paper covers the history of the ground-water investigations up to the fall following the filling of the reservoir and, in his closing discussion, he has brought the history up to date. For these reasons, any account of the studies on the south flood plain, by the writer, would be superfluous. These studies were made under the direction of Mr. Gongwer and many of the methods used were originated by him. They included investigations of ground-water levels as related to rainfall and reservoir elevations, extensive drilling, sounding, and contouring of the water table, measurement of flows from flowing wells, the use of fluorescein and geophysical apparatus, chemical analyses, comparative temperature readings, and correlation of water levels with barometric pressures. The studies yielded much valuable and exact information which, the writer believes, points to the following conclusions.

There is an inflow into the area, downstream from the central part of the south embankment, of something more than 140 gal per min. Drilling and ground-water studies indicate the flow to be rather deep, through seams in the foundation rock, rather than through the overlying alluvium. The seams in the rock and the permeable portions of the alluvium are all interconnected, however, as closure of the valves of the flowing holes produces a quick rise in practically all the holes in the vicinity. The underground reservoir must be comparatively tight and of limited size; otherwise, hydrostatic pressure could not be built up so rapidly in the holes. Since a flow of 140 gal per min reduces water levels as much as 7 ft, the inflow must be comparatively small. Before the holes were drilled which tapped the water beneath the flood plain, the water must have accumulated slowly, under pressure, being held down by the cover of impervious clay that blankets the flood plain. Its outlets were obviously much restricted, so that instead of moving away freely it rose high in the alluvium.

The most obvious source of the water is, of course, the reservoir, although the possibility that it is derived from the highland beyond the abutment has also been considered. A divergence between its chemical composition and that of the reservoir water lends weight to the belief that it may be ground water. However, the difficulties involved in the transmission of ground water from the abutment to the center of the flood plain, under sufficient pressure to cause it to flow out at the surface, appear so great as practically to preclude the abutment area as a source. Ground water drains away most freely near the abutment and stands considerably lower there than in the area of flowing wells. This condition could hardly exist if water were moving from the abutment to the wells.

In the writer's opinion, the reservoir is entirely responsible for the flow, even though analyses of the water reveal it to be more similar to ground water than to reservoir water. It must be remembered that the alluvium of the flood plain contained a large volume of water previous to the filling of the reservoir. When the reservoir was filled, pressure was put on this store of ground water so that it was forced through some small crevice or seam under the axis of the dam, in the foundation rock, which had, somehow, escaped complete closure. The water accumulated under pressure, downstream from the dam, until it was tapped by drill holes and flowed from the surface. Since the flow is relatively small, the ground water being pushed ahead of the reservoir water has not yet been exhausted and may not be for some time. If this hypothesis is correct, the contents of the water flowing from the holes will gradually approach that of the reservoir water and will ultimately approximate it.

The flow downstream from the dam is negligible and, by allowing the water to drain freely, ground-water levels may be held below the surface of the ground at all points. The south embankment may be considered unusually tight and effective. There is no likelihood of piping occurring below it.

Stream flows in the valley below the dam, in which the inlets to the large cave are located, have been carefully checked, and thorough observations of inlets and possible areas of seepage made. These indicate that no leakage whatever is occurring through the cave, which was blocked with a concrete bulkhead as a precautionary measure.

Mr. Marsh has pointed out the danger of gradually increasing leakage through limestone foundations as a result of progressive enlargement of seams and channels in the rock. The development of such leaks has presented a difficult problem at numerous dams. The Great Falls Dam, on the Caney Fork River, in Middle Tennessee, is a notable example. When the dam was built, in 1916, and for a good many years thereafter, leakage was relatively insignificant. As time passed, seepage through the narrow limestone divide that separated the reservoir from the river valley below the dam became steadily greater. The leaks increased in size and many new ones appeared. At present, water flows freely from a series of openings that extend from the dam to a point nearly a mile downstream from it.

However, careful studies indicate that the augmented flow has been produced by the washing out of clay filling which almost plugged an extensive system of previously existing solution channels, rather than by any further solution of the limestone itself. Numerous channels existed, but they had been formed by the extremely slow solvent action of water, operating over hundreds of thousands of years. It is interesting to note that now, since most of the clay has been washed out of these channels, the rate of increase of the leakage has markedly diminished.

At every dam with which the writer is familiar, where enlargement of openings in a limestone foundation has occurred, such enlargement seems to have been caused by the washing out of an accumulated filling rather than by actual solution. Although a well-consolidated limestone is a soluble rock, in comparison to most other rocks, it still is likely that many thousands of years must elapse before it would be affected appreciably by solution. If all seams and openings in a limestone foundation are thoroughly cleaned, so that all clay, sand, or other unconsolidated material which may be present is effectively removed and the cavities, consequently, can be completely blocked with grout or concrete, no troublesome leaks should develop from subsequent solution of the foundation during the life of the dam.

The writer appreciates the interest of those who have taken part in the discussion of the paper. Their comments have brought out various pertinent features relative to limestone foundations.

VERNE GONGWER,[33] M. AM. SOC. C. E. (by letter).—To those who have taken occasion to discuss this paper, the writer wishes to extend his thanks. Mr. Leonard corroborates the fact that the grouting under the second-stage and third-stage cofferdams was greatly advantageous to speedy and economical prosecution of the work. Had the work in the spillway (or second stage) cofferdam not been performed in record time during the summer and fall of 1937, the final operation of the generating units, and to that extent some phases of national preparedness, would have been impeded.

The writer, however, disagrees somewhat with Mr. Leonard as to the Guntersville Dam site being a "supersite." It can be agreed that probably "Few dam foundations in limestone are found that are better than that under Guntersville Dam," if taken in the literal sense, since nothing but very sound limestone and comparatively few uneradicable seams and caverns, solidly and tightly filled with good concrete or grout, were left under any part of the dam or any of its appurtenances. The large amount of inferior and undermined rock which it was necessary to excavate to make this a reality, plus some good rock necessarily excavated for the deep draft tubes, less an approximately equal quantity wasted at various points, was sufficient to riprap completely the 35-ft high river banks for about one-half mile downstream from the dam on both sides of the river, and to a thickness of 6 ft to more than 12 ft.

Where Mr. Leonard states "In the entire length of 4,000 ft, only two areas totaling about 600 ft in length showed signs of being cavernous or unsound; and these areas were fairly close to the surface," he possibly refers only to the length of two peculiarly cavernous areas along the axis under the south earth dike from station 31 + 65 to 37 + 10 and from station 39 + 50 to 40 + 15, totaling 610 ft (see Fig. 48). After costly deep excavation between double rows of interlocking steel sheet piling, these areas were found to be totally undermined and very cavernous, with partly filled seams of considerable lateral extent defying exploration. They occurred as high as 12 ft, directly under the line of the cutoff wall, with many branching arms and broad open seams, ramifying both upstream and downstream. It cannot be quite agreed that they were "fairly close to the surface," or at least not in the sense that they could be easily reached and corrected, since some of them lay as much as 25 ft below the top of solid rock and 60 to 65 ft below the ground surface.

In addition to the 610 ft of major caverns in this one local group, found upon opening up the south cutoff, Mr. Leonard has apparently overlooked other definitely cavernous areas—(*a*) 200 ft along the north cutoff foundation (see Figs. 30 and 32); (*b*) approximately 320 ft along the axis from the center line of the lock to and including pier No. 6 of the spillway (see Figs. 33, 34, 36, and 65); (*c*) 965 ft from station 22 + 00 to station 31 + 65, most of which required more or less trenching into the rock, and where water flowed copiously from seams and grout holes, also fluorescein from several hundred feet away; (*d*) approximately 200 ft under the mountain slope, constituting the south abutment, requiring much deep drilling and grouting to insure watertightness of mud seams; (*e*) small water-bearing caves leading into the sump common to the lock and spillway cofferdams (see Fig. 65); (*f*) an extensive undermined area with solutionized horizontal and vertical joints beneath and downstream from the north one third of the spillway apron where all exposed rock was dowelled to the sound rock below the shale, filled and grouted; and (*g*) small caverns near the junction of the tailrace wall with the southwest corner of the power house. Exclusive of the south abutment, the foregoing shows 2,100 ft along the axis of the total 4,000 ft-length of the dam, as cavernous and undermined, necessitating major correction. Certain of the other items caused flooded cofferdams, excess equipment breakage, and much delay and expense.

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[33] Tacoma, Wash.

FIG. 65.—CAVERNOUS AND UNDERGROUND SOLUTION SEAMS ENCOUNTERED IN EXCAVATING FOR A SUMP IN THE LOCK COFFERDAM

Many long, deep, vertical solution channels in the remainder of the total length of the river-bed section, embracing most of the spillway and power house, required painstaking and expensive remedial measures for opening, cleaning, and filling, of which Fig. 66 is typical. In addition, it was considered necessary to drill one or more 36-in. holes on each vertical seam and fill with concrete for blocking the seams. Also in the two 36-in. holes drilled and concreted under each spillway pier, the designing engineers required embedment of heavy vertical steel and column hoops to increase shearing resistance of the shale seam. Fig. 39 is misleading; it was selected expressly to show one local perfectly sound area where typical river-bed erosion (as distinguished from solutionizing) could be seen clearly. Practically all other foundation areas in all three cofferdams required more or less extensive correction or excavation to the general shale seam. At a conservative estimate, a concrete cutoff to shale, for the 2,100 ft of definitely seamy and cavernous rock, would have cost about $350,000, and the disturbance of adjacent rock due to the blasting in the trench and other indirect and consequential costs could easily have doubled or tripled that sum.

FIG. 66.—VERTICAL SOLUTION SEAMS UNDER INTAKE AND POWER HOUSE, CLEANED AND NEARLY READY FOR CONCRETE FILLING

As to whether several other sites could have been developed, feasibly or economically, near the head of Wheeler Pool, reference is made to the paper by Mr. Ross (see heading "Selection of the Site"), in which the history of the search for a dam site in this particular reach of the stream is summarized. From the record related it would appear that the "Coles Bend Bar" site (later known as the "Guntersville" site) was practically a "ground hog case," there being no other feasible site found after diligent surveys and explorations.

Eight sites were investigated at considerable cost, seven of which were eliminated as totally unsuitable. There evidently was no other economically or physically feasible site within the limits of the reach of the river available for building a dam to fit properly between the newly completed Wheeler Dam and the older Hales Bar Dam, approximately 80 miles upstream, completed about twenty years previously (see Fig. 22). It is doubtful whether the locating engineers fully realized either the many disadvantages of the site or the few redeeming features which later developed.

Mr. Leonard is probably correct in the assumption that the shale seam outcrops some distance below the dam, but there was no direct evidence to that effect. On the other hand, it is quite possible that the dip may slightly reverse shortly in that direction. Whether or not this shale seam does outcrop below the dam, subsequent contract dredging operations for channel improvement below the dam have revealed very extensive and troublesome deposits of massive residual boulders, produced by the processes of solution, such as are seen in Figs. 27, 28, and 42 and actually excavated; also at least one deep solution channel more than 50 ft below the stream bed was discovered. It is not believed, therefore, that a location in this area would have been advantageous. Also, there was nothing within the next 4 miles high enough or near enough to form a right abutment for the dam.

Honeycomb Cave did not add to the desirability of the Guntersville site (see Fig. 26). This cave, with upper and lower passages and multiple mouths considerably below pool level, was surveyed for approximately one-half mile and traced by fluorescein about 2 miles farther to a number of sinkholes downstream from the dam. Several of these sinks had earth rims slightly above pool level and bottoms in solutionized rock somewhat below pool level. Direct connection from the unsurveyed part to the river downstream from the dam was a distinct possibility.

The only feasible place where this intricately branching cave could be blocked off successfully was finally located about 700 ft inside of the mouth. The consultants and TVA engineering heads were considerably concerned with regard to the possibility of other thinly masked outlets, branches, or connections under the valley floor, or possible connection with one of the several springs below the 595-ft contour. Upon completion of the plug (at considerable cost), the cave was filled rapidly, by the water of the small creek therein, to the floor of the upper passage, 6 ft above pool level; and during approximately two years no leaks were discovered below the dam. Had such potential leaks occurred, and had no good place been found to plug the cave in advance, it would have been very costly and possibly very unfortunate, as construction of the dam was already well advanced. Great credit for the careful survey of this cave and the location of the only possible point for blocking is due Mr. Ross, and possibly others of whose participation the writer may not be aware.

The foregoing should not be construed as disparaging the preliminary and planning work of the engineers of either the Army or the TVA, or of Mr. Leonard's valued opinions or able discussion. It is intended merely to show that Guntersville was a definitely difficult site and to call attention to the narrow margin of feasibility, and necessity for grasping every favorable feature, involved in planning and constructing a chain of dams on such a river as the Tennessee—all of which redounds to the credit of the TVA engineering organization, in general.

As stated by Mr. Leonard, it is doubtful, in any case, if deep grouting below the shale seams could be dispensed with properly, in that, despite all the care and expense incurred, there yet remains in some minds a question as to whether or not the source of the water, producing the pressure phenomena in the rock seams below the dam, does not come from uncorrected channels connected with the reservoir. In this connection, the writer has recently been apprized (without opportunity to gather or verify further data) of the existence of a theory, approaching fact, that there is artesian pressure under the entire Tennessee Valley floor. In that case, the "ground-water hump" phenomena would be readily explained. It is regretted that a few very deep holes could not have been drilled within a mile or so downstream to check this theory.

Certain of the grouting and cavern filling methods are believed to be actually unique and more or less original, such as (1) the drilling and filling with concrete of "interlocking" 36-in. holes; (2) the filling of horizontal seams of some height with fine-aggregate concrete through drill casings driven into them from the original ground surface; (3) the concreting of other seams through 36-in. drill holes, between forms consisting of tiers of concrete-filled and sand-filled gunny sacks; (4) the jetting of overburden into caverns and "lost" ground downstream of the cutoff in such a manner as to balance the pressure of puddled backfilling between the two rows of steel sheet piling; and (5) the successful and advantageous sealing of cellular steel cofferdams by grouting of underlying open seams in limestone or other material by drilling down through the center of the cells into the seams or between and under residual boulders.

Notwithstanding an early report of the board of consultants to the general effect that cutoff provisions possibly would not be necessary at Guntersville Dam, the positive need for them was later recognized and authorized by the responsible engineering heads. During an early inspection were viewed: (*a*) The terrane, test pits, and spoil; (*b*) compartment boxes of samples of silt, sand, and gravel obtained by sinking 24-in. casings to rock where ground water had defeated open-pit methods; (*c*) the logs of tests with a small pump on a line of test pits from the river bank to the south bluff; and, more particularly, (*d*) the record of one of the pits not far from the south bluff where the pump (125 gal per min) could lower the water only 0.2 ft per hr. At the latter point sand ran under the sheathing of the pit and was pumped out copiously. It is probable that these data were not available at the time of the report mentioned.

It is worthy of note that this pit was adjacent to the area where the most serious and but partly filled seams and caverns were later encountered and required major corrective measures. The sample boxes for this pit, and an adjacent pit to the south, revealed materials close to bedrock ranging from fairly clean sand and gravel to large cobbles. Significantly, a characteristic valley in the ground-water contours now exists in this vicinity. These materials and many definite iron-stained open water courses therein were revealed in the excavation along the cutoff piling. As to unfavorable conditions to be subsequently encountered below the surface of the rock itself, none of the preliminary borings of the U. S. Engineers or of the TVA, on the customary rather wide spacing, gave more than slight indications, and these indications were interpretable only after detailed explorations and construction were well advanced. That these conditions were serious, and that they required positive and expensive remedial measures, with results equivalent to those adopted, is obvious. Apparent failure to recognize or to cope adequately with assumedly similar rock conditions at the old Hales Bar site resulted in extensive subsequent leakage. Had not the seriousness of the conditions under the flood plains at Guntersville been recognized, and had not the extensive and expensive measures been adopted which were used, it is the considered opinion of the writer, concurred in by a number of experienced engineers familiar with the site, that piping and underflow would certainly have resulted, under very similar conditions (but on a much larger scale and with much more serious results) to that which occurred at a smaller dam, near Reading, Pa.[34]

The great extent of large residual boulders was evident only after the sheet piling had been driven and after the irregular seating of the piling had been investigated by exploratory excavation. The findings necessitated an arduous program, important both as to first cost and as to effect on construction schedule and indirect cost.

One of the main points that the writer desired to emphasize was that, in such erratic and temperamental terrane as soluble limestone (and, in fact, at most proposed dam sites of any nature), preliminary investigations, while avoiding, if possible, unnecessary expense, should go beyond bare customary practice and should be sufficiently extensive to obviate such surprizes as were experienced, and such changes of plans as were necessitated, at Guntersville Dam. Preliminary investigations of this scope should contemplate, and include from the outset, sufficiently complete and effective methods, and sufficiently heavy-duty pumps, drills, and other equipment, to obtain the necessary and vital results and data. The corrective measures necessitated meticulous attention. As construction engineer, Mr. Leonard is to be highly commended for the final satisfactory and watertight results. The writer agrees almost entirely with the remainder of Mr. Leonard's discussion and wishes to thank him for submitting it.

Although the writer is of the opinion that limestone seams "solutionize" very, very slowly (requiring perhaps 100 to 500 years for any appreciable or dangerous enlargement to develop), the remarks of Mr. Marsh are particularly well taken. In principle, they are definitely pertinent to any dam site, in any formation or kind of rock, and to all other engineering structures as well.

In continuing the observations of the ground-water wells, as routine procedure, and for the purpose of clearing up some uncertainties in properly depicting the ground-water contours, several additional observation wells were driven after the paper was presented. It can be readily seen and understood that the problem of locating wells for maximum information is a process of trial and error, and that always some previously driven wells might well be dispensed with and a few additional wells could yet be desired. Also, at the end it is usually regretted that the final nearly ideal system of wells could not have been foreseen and driven in the beginning so that a long, complete record could be had. Frequently, also, early conceptions of the behavior of the ground water are considerably revised as the study progresses. It is practicable in the light of the later findings and observations to return to a review of the early days of the work (when fewer wells, less satisfactorily located, existed) and, with care, reinterpolate and reinterpret the early ground-water maps, thus

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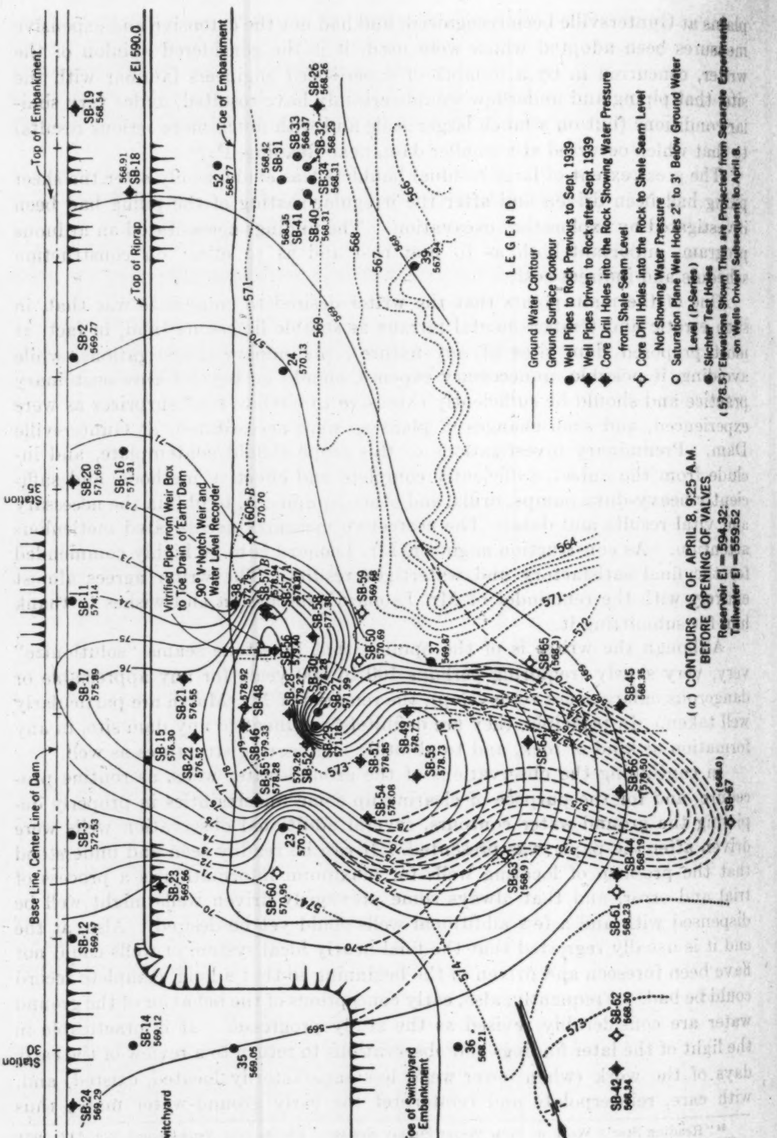
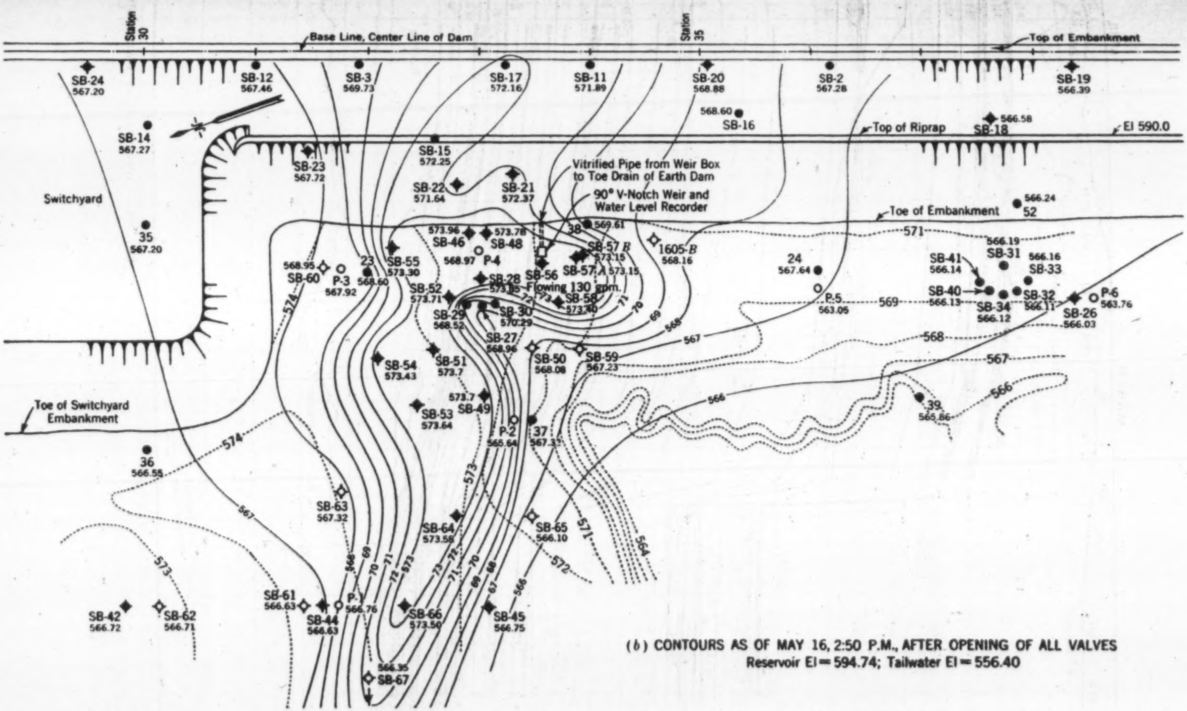
[34] "Reading Starts Work on New Water Supply System," *Engineering News-Record*, Vol. 113, 1934, p. 323; "Unmapped Eroded Limestone Complicates Corewall Job," by Farley Gannett, M. Am. Soc. C. E., *loc. cit.*, Vol. 115, 1935, p. 328; and "Caverns Under Dam Corewall Set a Nice Repair Problem," by Farley Gannett, *loc. cit.*, Vol. 116, 1936, p. 492—see also p. 502.


Base Line, Center Line of Dam
Top of Embankment
Top of Riprap
Toe of Embankment
El 590.0
Station 30
Station 35
Switchyard
Toe of Switchyard Embankment
Vitrified Pipe from Weir Box to Toe Drain of Earth Dam
90° V-Notch Weir and Water Level Recorder
SB-24 569.29
SB-14 569.12
SB-12 569.47
SB-3 572.53
SB-15 576.30
SB-17 575.89
SB-11 574.14
SB-20 571.69
SB-16 571.31
SB-2 569.77
SB-18 568.91
SB-19 568.54
SB-23 569.66
SB-22 576.52
SB-21 576.55
SB-55 578.28
SB-46 579.33
SB-48 578.92
SB-52 579.26
SB-29 571.18
SB-28 579.27
SB-30 574.28
SB-27 571.99
SB-36 579.20
SB-58 577.38
SB-57 A
SB-57 B
38 572.75
52 568.77
SB-31 568.42
SB-33 568.37
SB-26 568.26
SB-32 568.29
SB-34 568.31
SB-40 568.31
SB-41 568.35
39 567.94
24 570.13
1605-B 570.70
SB-59 569.68
SB-50 570.69
37 569.87
SB-65
SB-45 568.35
SB-51 578.85
SB-49 578.72
SB-53 578.73
SB-54 578.08
SB-64
SB-66 578.50
SB-67 (568.0)
SB-44 568.19
SB-63 568.91
SB-61 568.23
SB-62 568.30
SB-42 568.34
SB-60 570.95
23 570.79
35 569.05
36 568.21
564
565
566
567
568
569
570
571
572
573
574
575
LEGEND
Ground Water Contour
Ground Surface Contour
Well Pipes to Rock Previous to Sept. 1939
Well Pipes Driven to Rock after Sept. 1939
Core Drill Holes into the Rock Showing Water Pressure from Shale Seam Level
Core Drill Holes into the Rock to Shale Seam Level Not Showing Water Pressure
Saturation Plane Well Holes 2' to 3' Below Ground Water Level (P-Series)
Slichter Test Holes
(578.5) Elevations Shown Thus are Projected from Separate Experiments on Wells Driven Subsequent to April 8

(a) CONTOURS AS OF APRIL 8, 9:25 A.M. BEFORE OPENING OF VALVES

Reservoir El = 594.32; Tailwater El = 559.52


Station 30
Station 35
Base Line, Center Line of Dam
Top of Embankment
Top of Riprap
El 590.0
Toe of Embankment
Switchyard
Toe of Switchyard Embankment
Vitrified Pipe from Weir Box to Toe Drain of Earth Dam
90° V-Notch Weir and Water Level Recorder
Flowing 130 gpm.
SB-24 567.20
SB-12 567.46
SB-3 569.73
SB-17 572.16
SB-11 571.89
SB-20 568.88
SB-2 567.28
SB-19 566.39
SB-16 568.60
SB-18 566.58
SB-14 567.27
SB-15 572.25
SB-23 567.72
SB-22 571.64
SB-21 572.37
52 566.24
35 567.20
36 566.55
38 569.61
SB-46 573.96
SB-48 573.78
P-4 568.97
SB-57 B 573.15
SB-57 A 573.15
1605-B 568.16
SB-55 573.30
23 568.60
SB-60 568.95
P-3 567.92
SB-56
SB-28 573.05
SB-52 573.71
SB-29 568.52
SB-30 570.29
SB-58 573.40
SB-27 568.96
SB-50 568.08
SB-59 567.23
SB-54 573.43
SB-51 573.7
SB-49 573.7
SB-53 573.64
P-2 565.64
37 567.37
24 567.64
P-5 563.05
SB-41 566.14
SB-31 566.19
SB-33 566.16
SB-40 566.13
SB-34 566.12
SB-32 566.11
SB-26 566.03
P-6 563.76
39 565.86
SB-63 567.32
SB-64 573.58
SB-65 566.10
SB-61 566.63
SB-44 566.63
P-1 566.76
SB-66 573.50
SB-45 566.75
SB-42 566.72
SB-62 566.71
SB-67 566.35
574
573
572
571
569
568
567
566
564
73
72
71
70
69
68
67

(*b*) CONTOURS AS OF MAY 16, 2:50 P.M., AFTER OPENING OF ALL VALVES
Reservoir El = 594.74; Tailwater El = 556.40

FIG. 67.—GROUND-WATER MAP FOR 1940, STATION 30 TO STATION 38, SOUTH EMBANKMENT

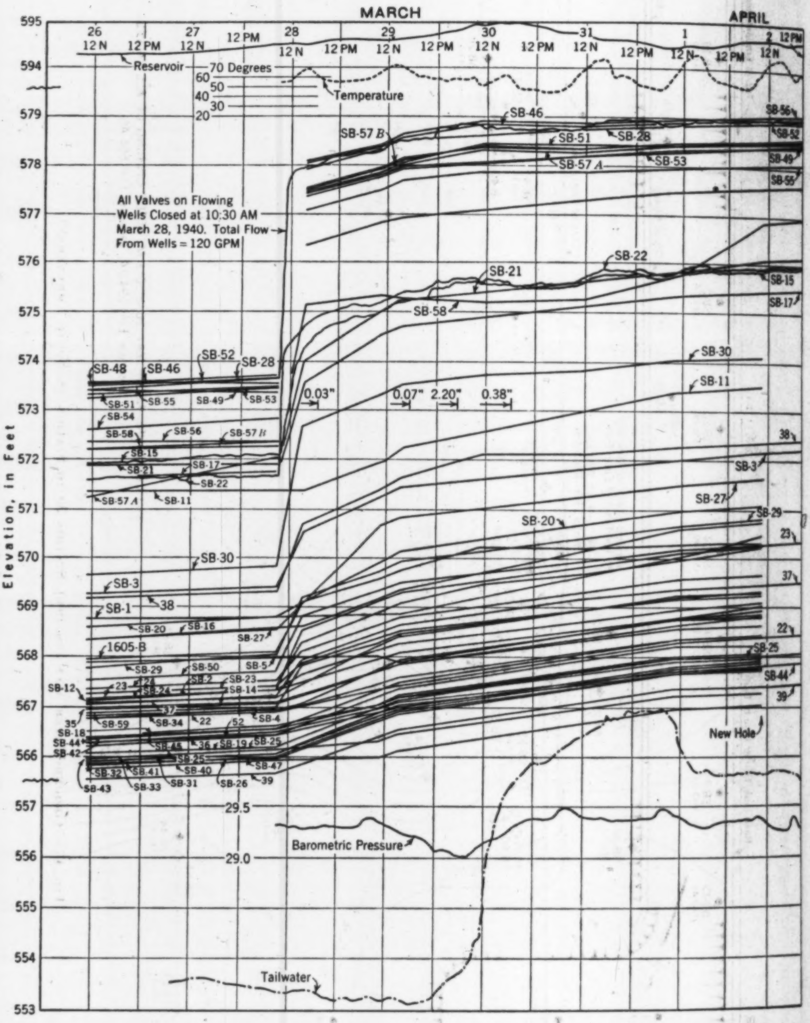
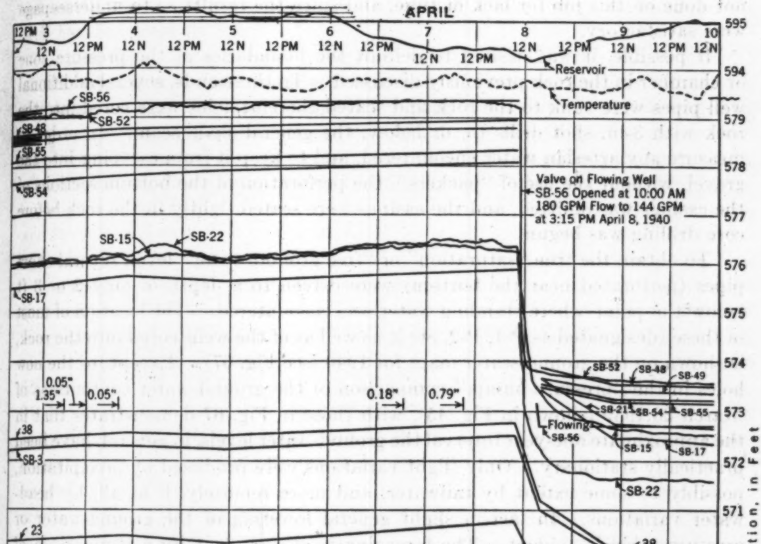


FIG. 68.—COMPARISON OF WATER LEVELS



IN WELLS DURING PRESSURE EXPERIMENT

leaving a more understandable record and gaining a much better idea of the conditions than, likely, was held during the progress of the work. This was not done on this job for lack of time, and since the results as to underseepage were satisfactory.

If possible, it was desired to delimit the boundaries of the pressure zone or chamber in the rock previously discussed. To these ends, several additional well pipes were sunk to the rock and seated therein; holes were cored into the rock with 3-in. shot drills to, or below, the general shale seam. In order to measure any artesian water encountered, and to keep it from escaping into the gravel, without the use of "packers," the perforation of the bottom sections of the casings was omitted, and the casings were seated tightly in the rock before core drilling was begun.

To obtain the true "saturation" or "free ground-water" level, several short pipes (perforated near the bottom) were driven to a depth of only 2 or 3 ft below the point where standing water was encountered. The location of most of these (designated as P-1, P-2, etc.), as well as of the wells cored into the rock, is shown on the ground-water maps for 1940 (see Fig. 67). Except for the new holes in the "pressure bump," comparison of the ground-water contours as of March 26,1939 (shown in Fig. 45), with those in Fig. 67 demonstrates that in the approximate one-year interval the ground-water levels, in general, have been practically stationary. Only slight variations were produced by precipitation, possibly to some extent by tailwater, and more remotely, if at all, by headwater variations. In fact, a slight general lowering of the ground-water or pressure table is evident. The foregoing is also evident by a comparison of the water levels shown for March 31, 1939, in Fig. 46, with those for the same holes as of March 26, 1940, shown in Fig. 68. In Fig. 68, closely grouped curves are omitted after April 2, and the holes plotted were chosen as representative of a group. Straight lines represent holes read at intervals. Continuous graphs of wells SB-22, SB-52, and SB-15 are taken from recording instrument records. No wet or soft spots or new seepages have appeared below the dam during this period. Contours of the position of the "free" water table or saturation plane have not been attempted from the few shallow wells, but comparison of the elevations shown in Fig. 68 vary from fairly close agreement between shallow and deep wells (as between P-3 and SB-60) to a difference of about 7.3 ft (as between P-2 and SB-49). A rather exceptional coincidence exists under normal conditions between P-1, SB-44, and SB-61 (shallow, to rock, and into the rock, respectively); yet the last two respond positively when the flowing wells are shut off, as in the pressure experiment.

The various phenomena connected with the progressive driving of the additional wells were very interesting and helpful toward an understanding of conditions and would assist in future similar investigations. However, a detailed description is not possible for lack of space. The first flowing well resulted from drilling hole SB-28 down into the rock with the intent of applying a Slichter determination to the flow in any rock seam encountered. The most significant phenomenon was that as each succeeding flowing well was encountered an immediate lowering of the water level in well SB-22 resulted, the total effect being observable from the several ground-water maps. All wells as drilled were equipped with tees, valves, and static riser pipes, from manipulation of which some very helpful data were obtained. All flowing wells were extremely sensitive to back-pressure from well SB-56, or whatever previous well had the greatest flow.

Fig. 68 shows the graphs of the water surface in all wells existing at the time of the experiment; headwater and tailwater elevation; barometric pressure; temperature; and date and quantity of rainfall. The latter, and doubtless to some extent the high tailwater, caused by the necessity of spilling water from the reservoir, created a slow rising trend in all wells, beginning 24 hr to 36 hr after the intital rise due to closing of valves. This trend continued until the moment of opening the valve on SB-56. It should be noted that the substantial and fairly rapid variation in headwater is definitely not reflected in the behavior of the wells.

As indicated by the graphs of the three automatic recorders, the behavior of the three wells, SB-15, SB-22, and SB-52, was remarkably similar and evidently is representative of the manually observed wells wherein the detailed variations were impracticable of observation. The recorder graphs of wells SB-15 and SB-22, which were driven only to rock, agree exactly, whereas that of well SB-52, which was cored down into the seams in the rock, does not quite agree as to every detail of variation. Quite obviously, none of them shows headwater influence, and rainfall influence is but slightly indicated. However, several sharp rises in all three automatic graphs are coincident with sharp drops in barometric pressure; and, also, the reverse is noted, being in all cases sharply coincident as to time. The water stage recorders and barograph produced automatic records; and therefore no personal error or influence is involved. One or two longer trends between water levels in wells and barometric pressure appear to be likewise related. Sharp tailwater variations do not seem to affect the graphs.

Barometric influence on the water levels in the wells, thus definitely established, would appear to indicate substantial tightness of the underground pressure chamber or seams, definite freedom from any but the slightest reservoir influence, and largely predominant artesian or remote water supply rather than any considerable supply from reservoir seepage even if it occurs through deep and circuitous channels. If any considerable part of the well flow were from the reservoir, the route would have to be long, very constricted, and circuitous; otherwise, barometric variations would be balanced between the reservoir surface and the water surface in the well casings and would produce no variation in the water level in the wells. Since pressure variations are transmitted practically instantaneously, even through long, very small, or capillary tubes, it would seem evident that the reservoir can have little more than capillary connection with the pressure area, making allowance for modified or dampened action in case the flowing water is supplied predominantly from artesian or other source. In the opinion of the writer, barometric influence is quite definitely indicated by the graphs, and comparison of barometric and water-level graphs therefore seems a very desirable and advantageous phase of such an investigation.

The writer's attention was called by Mr. Harris to the apparent influence of temperature variations on water levels in the wells. There appears to be some similarity between the temperature graph and the well graphs in a tendency to rise during the late forenoon and then to level off in the afternoon. On certain days, however, marked temperature rise is not accompanied by rise in the wells, and in certain of those instances the barometric influence seems to have been the controlling factor and to have been opposite in effect to temperature. It is probably true that a marked temperature rise, when the water is standing several feet above the ground in the pipes, would warm up the top of the water column and cause a slight rise in the water surface independent of reservoir level, which does not, from the graphs, seem to have any definite influence on the wells, either sharp or long range. However, the influence of the barometric pressure variations seems to be the stronger and quite sharp. No doubt some combination of the several influences affects the height of the water columns in the well risers. Temperature would operate independently of the source of the pressure water, whereas barometric pressure effects would be operative in discernible amount only where the source is comparatively remote, considerably constricted and circuitous, or artesian, and the pressure chamber substantially tight. All evidence indicates that the barometric influence is much the strongest of the several possible influences, and definitely indicative of predominantly artesian supply.

Fig. 67(*a*) shows the location of all wells in the study area existing at the beginning of the experiment, the water levels in these wells, and the corresponding highest ground-water (or pressure water) contours immediately prior to reopening of the valves. A "before map" preceding the experiment is not essential since it would be practically identical with the "after map," Fig. 67(*b*). In well SB-56 elevations were taken just before opening the valve at 10:00 a.m. In Fig. 67(*a*) holes with elevations in parentheses were drilled after this date. The water-pressure contours have been shown by solid lines as they were justifiably interpretable on that date (April 8) from the then existing holes; also by dotted lines as projected or extended by careful estimation from full field acquaintance to correspond with the certain behavior of wells SB-63, SB-64, SB-65, SB-66, and SB-67, driven subsequent to the experiment. In a study of this kind, this illustrates strikingly the limitations and errors introduced by an inadequate number of exploratory holes. Probably additional wells near SB-67 would have shown that the pressure area extended still farther downstream (or westerly). It is believed that the zone was quite thoroughly defined in all other directions.

Fig. 67(*b*) shows the ground-water (or pressure) contours as of May 16 at 2:50 p.m., some days after the opening of all valves. The elapsed time interval was necessary to utilize the observations of the last and final wells driven at the site. Fig. 68 shows the wells practically returned to normal on April 10. In general the rise in all pipes during the experiment was at approximately the same ratio as in the preceding experiment (see Figs. 48 and 49). On April 8, upon opening of all valves at 10:00 a.m., well SB-56 discharged 180 gal per min. This dropped to 144 gal per min by 3:15 p.m. and shortly returned to normal (120 to 130 gal per min).

As "one well led to another" in searching out the boundaries of the pressure area, the size and shape of this area changed considerably from earlier impressions. It is noteworthy that the wells registering the highest normal water levels, with all valves open, lie in a comparatively straight line through well SB-56 and approximately station 35 + 00 of the dam axis. Well SB-67 was located with this in mind but failed to strike the solution channel or pressure seam, probably by feet or possibly only by inches. It also will be noted that wells Nos. 1605 B and 19 (fluorescein wells), shown in Fig. 45, indicate a drainage or seam axis of the same general direction, passing through approximately station 35 + 25. However, this latter seam or drainage system evidently was at a different level from the one previously discussed, and apparently not connected with it, as it will be noted in Figs. 45 and 67(*a*) that well 1605 B is not affected by pressure rise during the pressure experiments. Also it is evident that this seam was effectively blocked by the cutoff. The wells active during these experiments evidently pierced a seam or system of seams at a different elevation or horizon. This is understandable since many of the solution seams at different levels follow the general direction of one of the principal directional axes of jointing.

Numerous caverns with important arms and seams branching from them were observed in excavating along the cutoff trench, some of which could have been more deeply explored and mapped, at some possible risk, had the pressure tests and the questions raised by them been foreseen at the time. Certain of these branch caverns lead off in the general direction of wells SB-56, SB-28, SB-49, etc. It is advised, therefore, in similar investigations, that all such tributary seams and arms, together with vertical seams and chimneys rising from these seams, be surveyed and mapped as completely as possible wherever and whenever encountered.

All questions as to the integrity of the dam and cutoff provisions having been answered satisfactorily, at the substantial conclusion of construction of the dam and power house about May 1, 1940, arrangements were made for periodic future observations in all ground-water wells and saturation-line wells in the rolled-fill sections of the dam; and a permanent installation was devised and constructed for measuring the flow of the key well, SB-56, which comprised the total known flow from the pressure zone. This installation consisted of a concrete box, with its top flush with the ground surface, comprising a stilling well with baffles and a V-notch weir, with vitrified pipe outlet discharging into the toe drain of the earth dam, and with an automatic water level recorder in the stilling well. All well pipes were cut off just below the surface, tamped about tightly with clay, to exclude possible free entry of rain water, and provided with screw caps for discouraging unauthorized tampering but permitting continued observation. None of these pipes would overflow upon uncapping, with well SB-56 flowing freely, and should they do so later, due to possible future changes, the observer, upon removing the cap, would lightly screw on a riser pipe of sufficient length to permit the necessary observations.

Before leaving the shallow "true" ground-water wells, and the saturation plane wells in earth embankments, all such holes were earth-augured for a foot or two below the bottom of the casing to insure free entry of ground water.

It should be mentioned here that extreme care should be used to ascertain that all observation wells are sensitive or "active." Some wells in tight ground appear very sluggish. However, many apparently "dry," inactive, or "lost" wells can be rendered active by auguring below the casing, punching out the plugged perforations with a nail in a stick, or "shooting" the bottom of the pipes with a quarter stick of dynamite. Many holes considered "dead" and hopeless frequently became "active" with such treatment. Unremitting care and perseverance are necessary to obtain complete and dependable readings and effective records. Seemingly almost everything possible conspires to prevent a complete, full record of water levels being obtained, and it is well to approach such an investigation with that idea and attitude; otherwise disappointments and poor records almost invariably result.

As the casing of well SB-56 was cut off at slightly lower elevation, to discharge properly into the weir box, well SB-22 dropped to El. 570.81, the lowest yet recorded for this well, thus permanantly insuring that there will be no upward pressure under the toe of the rolled earth fill.

The flow of well SB-56, as of November 25, 1940, had decreased to 85 gal per min with the reservoir only 1.1 ft below normal level of 594, which further seems to indicate that at least a substantial part of the total flow does not come from the reservoir since greater fluctuations than this have occurred without affecting the water levels in the wells appreciably. By long-time observation of the flow from well SB-56, if effect of heavy rains, barometric variations, etc., can be properly compensated for, some interrelation between reservoir level and the wells may of course yet be proved. However, there seems to be much greater weight of evidence that there is little connection.

The system of wells drilled into the rock seams, particularly the key well SB-56, obviously situated so as to tap the "main stem" of the pressure system of seams below the dam, with weir box and recorder, is believed to constitute, for an earth-fill dam over seamy and cavernous limestone, an ideal arrangement of drain holes or pressure relief holes, equivalent to that generally considered ideal for a concrete dam.

It is possible that a part, or the whole, of this study and investigation, particularly now that the satisfactory outcome is known, could have been considerably curtailed, but the writer is strongly of the opinion that, where an investment of $25,000,000 to $30,000,000 is involved, the comparatively small cost of such an investigation is definitely and fully justified, and to omit it, in view of past and recent history of dams and other engineering projects, should be considered as not good engineering practice.

In his discussion Mr. Moneymaker notes the disproving of the concept stated to have been long held by geologists to the effect that cavities may not be formed below the water table. The writer has been curious as to the line of reasoning, or experience, which originally gave rise to this concept. From an engineering viewpoint it would seem evident that ground water, in constantly seeking its level, lower outlets, etc., would frequently circulate through joints, bedding planes, or other orifices or pervious strata far below any existing or former free water table. The writer would think that such a theory would have long since been disproved by such known facts as the leakage channels under Hales Bar Dam below the original stream bed, the "lost" rivers of many parts of the United States, the many rising springs which apparently come from channels below the water table, etc. In the writer's opinion, drill holes, adequately deep and numerous, are the main evidence upon which final dependence can be placed in structures of major importance.

Quite properly, Mr. Moneymaker stresses the great advantage of early recognition of all conditions that threaten subsequent leakage, and the "cost and difficulty of stopping, or even appreciably retarding, leakage that has developed in limestones or dolomites under and around a large dam." The writer has frequently noted what is thought to be considerably too much complacency among engineers in regard to the possibility of fixing leaks after they once start. Most sites are capable of fairly certain diagnosis upon a thorough study of an ample number of carefully located drill holes and a study of the behavior of a reasonable number of ground-water observation wells, etc., plus perhaps a certain amount of practical engineering and geological intuition. Even in loosely, partly filled, and widely ramifying limestone caverns and seams, an appropriate amount of cost and time in previous investigation, grouting, and blocking will remove all reasonable possibility of serious leakage, and similar methods will diagnose and prevent (as described in this Symposium) serious leakage through pervious overburden. However, to further Mr. Moneymaker's thought, after several thousands or hundreds of thousands of acre-feet of stored water have torn through these same channels, due to having been left partly or totally uncorrected, it is certain to be a much different and much more serious matter.

Mr. Moneymaker appropriately mentions the early suspicion with which the Guntersville (Bangor limestone) formation was regarded and the considerable leakage under the only previous dam founded upon this formation (presumably Hales Bar Dam already mentioned by the writer). He also mentions that the largest caves in the entire TVA area occur in this formation. Mr. Ross, and certain associates, explored some twenty-five or thirty such caves in the Guntersville reservoir walls. One of these, Honeycomb Cave, completely by-passed the right abutment below pool level. Except for fortuitous circumstances and conformation, this large branching cave might have defied blocking-off operations, even at excessive cost. Great subsequent cost was incurred at Hales Bar, with only partial success, because of caverns and seams which were probably not so large. Another cave of almost comparable size, buried under the south cutoff, was discovered only after construction was well advanced and after many staff consultations had been held as to the advisability of not doing anything further than to rely on the steel sheet piling already driven.

The foundation problems, which Mr. Moneymaker (partly quoting Mr. Ross) describes as being serious only where there were solution cavities, actually involved, in addition to a complete steel sheet pile cutoff through the pervious gravel, about 2,100 lin ft, in the rock, of the approximately 4,000-ft length of the dam axis. Whereas the remedial treatment in the last analysis might be characterized as mainly excavating to good rock and grouting, it actually involved a number of extensive and highly expensive construction operations to remove masses of boulders and get down into the rock and correct the seams and caverns. All these were possible only with heavy pumping and progressive backing out, while progressively backfilling with what Mr. Moneymaker apparently refers to as "impervious material." In all cases this material was solely a good grade of concrete and grout. This work was anything but simple and inexpensive as might possibly be inferred from the brief reference by the discusser. It is possible that Mr. Moneymaker writes only in the geological sense, without reference to the extremely costly preparatory and coincidental construction and permanent corrective work.

It is noted that Mr. Walter found that a fine sand could be added to neat cement grout (evidently without admixture) for injection into solution cavities by means of the usual piston-type pumps. There has been some experience described where sand-cement grout has produced excessive wear on the pumps, but this effect apparently was not sufficient to cause Mr. Walter to comment thereon. Also, it is noted that Mr. Walter apparently leaned rather decidedly toward pumping grout into a hole as long as it conceivably could do any good, rather than to worry too much about possibly wasted grout, since a closely parallel hole frequently failed to connect with the usually irregular open area under treatment. The writer is in full accord with this general policy and has occasionally wished some "prolific" hole had not been allowed to "freeze up" slowly, especially after subsequent holes failed to find the irregular system of open seams. Mr. Walter's experience also proves the tight results obtained by regrouting, where necessary.

The writer agrees fully with Mr. Prouty's belief that projects involving great expenditures and responsibilities should have every possible check as to permanence and safety. Mr. Prouty's suggestion as to deep artesian effect is also very interesting.

Mr. Feld very properly suggests that it would aid the reader if the total cost of exploratory work and grouting for each of the dam projects was given. If anything beyond generalizations are to be attempted, it would be necessary to make a detailed study of each job in order to compare efficiency, results, etc., of different methods, as cost keeping and other procedures vary more or less in spite of standardization.

A rather complete ground-water study, such as was instituted almost from the beginning at Guntersville Dam, will, in large measure, answer Mr. Feld's question as to what changes downstream of the dams are to be expected from complete stoppage of the subterranean flows. However, since permanent tailwater will practically coincide with original river levels, conditions below the dams should change very little.

No admixtures of any kind were used with cement grout at Guntersville Dam at any point where watertightness or permanence was sought. The writer has grave doubts as to the wisdom of using any known and economically or practically feasible admixture in connection with any permanent part of structures of such importance and magnitude. A certain amount of fine sandy silt-cement mixture was used at Guntersville Dam in an attempt to fill certain rock cavities upstream and downstream of the cutoff (the latter already made tight and permanent by various methods described, all using a good grade of concrete or neat cement grout) of the south earth embankment to obviate possible cave-ins. The cement in this case was more or less of a gesture toward slightly more insurance, if possible, that the material, which was the natural material of the flood plains, would "stay put." When, similarly to the experience reported by Mr. Walter, the difficulty of finding the ramifying seams, with holes placed where the seams were estimated to be, became apparent, and a question of watertightness also became involved, the admixture of sandy silt was discontinued and only pure cement grout was used thereafter.

The writer agrees with Mr. Minear on much of his discussion, particularly that a larger number of the smaller diameter, and usually cheaper, grout holes generally will result in the most efficient and economical over-all results. It is found, as also reported by Mr. Walter, that the bulk of the grout usually is taken by a comparatively few holes which happen to pierce the seams or achieve advantageous openings in them. Also, frequently, due to interconnection, several holes, which often may be large-diameter expensive ones, may be wasted if drilled previously.

FIG. 69.—CEMENT GROUT THAT HAS IMPREGNATED SAND, GRAVEL, AND MUD (HYDRAULIC CELL FILLING AND RIVER-BED MATERIALS)

Mr. Minear's experience with grout coming in contact with seam filling material is noted. The writer has exposed certain areas where pure cement grout came in contact with dredge filling of gravel, sand, and mud as in Figs. 37 and 69. Subsequent opportunity for close examination revealed that the cement had everywhere retained its integrity completely, shoving the mud into pockets and interstices, and occluding it, so that the mass was hard and watertight, resembling concrete, with frequent isolated pockets of foreign matter. It also followed and compacted many white seams in residual clay, forming series of irregular watertight cement cells.

After all, it is a matter of engineering judgment as to method, probable results, and how much money should be spent in the initial operations of drilling and grouting. Previously cited papers on this subject, published in 1934, 1935, and 1936, are believed to be illustrative of some of the points discussed.[34]

Acknowledgments.—Grateful acknowledgment is made here to Henry D. Harris for unremitting effort, interest, and care in gathering data and for assistance in preparing the part of this paper dealing with the ground-water investigation, and to Mr. Ross for a careful study of all geological features and assisting the writer to a better understanding of them.

PORTLAND P. FOX,[35] **JUN. AM. SOC. C. E.** (by letter).—Mr. Marsh's warning to others who may attempt to build a dam on a soluble limestone foundation is well fitted into these discussions. Not all dam sites on limestone are necessarily bad, but any dam site on a limestone foundation should be regarded with much suspicion until it is proved to be satisfactory.

Perhaps too much stress cannot be put on the need for closely spaced and deep exploratory core-drill holes in a limestone foundation for a proposed dam. At least ten times as many core-drill holes may be required to prove a dam site on a limestone foundation as on most other types of rock foundations. In the TVA region a number of possible dam sites on limestone had been explored, previous to the creation of the Authority, by a few widely spaced and shallow core-drill holes, and most of the sites appeared to be nearly perfect from these few holes; but on closer drilling and with deeper holes many of them proved to be very unsatisfactory. It is far better to explore a dam site on limestone thoroughly than to do a halfway job and encounter many costly surprizes after construction has started.

Mr. Marsh's discussion in regard to a dam founded on a soluble limestone as a doubtful safe long-time proposition is supported by a number of cases in which dams on limestone have developed serious leakage a few years after completion. In all cases it appears that the cause of the leakage has been due to the washing out of clay, sand, and other soft material that partly filled the pre-existing cavities or solution channels in the limestone and not to any rapid solution of the limestone itself. It should be remembered in regard to rates of solution that concrete is more soluble than most limestones.

If all of the cavities are thoroughly cleaned and completely plugged with concrete or grout, as they were at Chickamauga Dam, no leakage can possibly occur. In order for a limestone to dissolve, water must come in contact with the rock; and there must be circulation of the water or it will become inactive. If all of the joints, seams, and other openings in the limestone under the dam have been sealed at some place so there can be no circulation under the dam, there can be no solution of the limestone. The entire success of any foundation on limestone under a dam may depend upon blocking all of the openings completely. If this is not done, as Mr. Marsh has pointed out, leakage (even unnoticeable at first) may develop over a period of years and may become serious.

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[35] Geologist, TVA, Spring City, Tenn.

At Chickamauga Dam a great number of drill holes have been located strategically downstream from the earth dams, and at several saddles in the reservoir, for ground-water observations. The ground-water conditions before and after filling the reservoir were observed and will be continued indefinitely by the Hydraulic Data Division of the Authority. Any changes in the ground-water conditions should be detected quickly.

James B. Hays,[36] **M. Am. Soc. C. E. (by letter).**—Mr. Simonds requests information as to the method used in grouting with asphalt or pitch. A 6-in. casing was seated in the rock and extended just above the surface of the ground, and 5½-in. holes were drilled into the cavity to be grouted. A 2-in. steam pipe, capped on the bottom, extended to within a short distance of the bottom of the cavity; and inside this was a slightly shorter 1-in. return pipe. The lower ends of the steam pipes had to be kept free to allow for expansion. The casing head was piped to receive melted asphalt between the 2-in. steam pipe and the outside casing. Steam was piped into the 2-in. pipe, which was brought up through the casing head, and was discharged from the 1-in. return which projected above the 2-in. pipe. Asphalt was heated in portable kettles of the type used by roofing contractors. These kettles have a built-in rotary pump which, in this case, did not stand up under long-continued use. Small, piston-type, air-driven pumps were made locally and used to force the asphalt into the cavity. These pumps had a cylinder 1 in. in diameter and were arranged so that the piston stroke could be varied from ½ in. to 6 in. The pumping rate varied from 1 to 2 bbl per hr. When asphalt is used the kettles soon become caked with carbon. Pitch does not cake.

In operation during cold weather the steam is turned into the heating pipe in the hole for a short period to warm it up before pumping of the asphalt or pitch is begun. Since the cavities to be grouted had water in them, a vent pipe was also provided at the casing header to release any steam pressure created during the introduction of hot asphalt. After the grouting is well started, the steam heat can be discontinued. During warm weather it was not necessary to use steam heat. If grouting has been stopped and it is later decided to resume operations, the steam is used to soften the material in the casing, after which grouting can continue.

Rapid pumping causes the asphalt or pitch to spread out in a thin layer or stringers. Slow pumping causes the formation of a large bulb, the outside shell of which is tough, being formed by cooling action in the water. The center remains hot and liquid, and there is considerable shrinkage when it finally cools off. Subsequent regrouting will eventually reduce the space left by shrinkage. For thin seams the rapid pumping gives greater coverage. To stop flows through large openings, the slow bulb-forming process seems to be more effective.

Mr. Simonds questions the stability of bentonite grout mixtures. At Chickamauga Dam it was used only for temporary construction grouting or for backfilling open cavities under the earth fill. Construction grouting consisted of blocking off flows of water into areas to be excavated later. This operation was highly successful. No dependence was placed on it for a permanent water seal. Small percentages of bentonite have been added to cement for permanent grouting on other projects, such as the Agency Valley Dam in Oregon, built by the Bureau of Reclamation. There is an increase of volume due to the addition of bentonite to cement, and a smoother mix results. It will crack or shrink on drying in warm air, but underground no doubt it will remain in good condition.

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[36] Constr. Engr., Kentucky Dam, TVA, Gilbertsville, Ky.

The clay-cement grout was used for temporary or backfill purposes only at Chickamauga Dam. However, it must be remembered that only certain types of clay will make a satisfactory grout. Topsoil is often used with cement to grout under concrete highways where settlement has occurred to bring the surface back to its former elevation.

Mr. Minear has emphasized several important points. The writer agrees with him that the rule of grouting until a hole refuses 1 cu ft of grout in a certain specified time is indefinite, inasmuch as it does not indicate the quantity of cement involved. The "second criterion" mentioned by him of pumping "to refusal at a pressure which is a fractional part (such as two thirds) of the allowable pumping pressure" might be misunderstood. Assuming that a hole is to be grouted to 100-lb pressure, his statement would mean to stop grouting with refusal at 66 lb. In conversation with Mr. Minear some years ago, the writer understood that it was his idea that pumping should be done at a higher pressure than that specified, in which case the refusal at a lower or fractional pressure would give the desired results. This conversation occurred at Hoover Dam during the construction period, and high-pressure methods were under discussion. The writer agreed with him. Other foundation conditions where pumping could not be done at higher pressures than the desired refusal, due to uplift, would require a modification of the rule. In either case the final water-cement ratio at refusal should be specified.

In regard to handling grout at a slow rate, the consequent settling out in the supply lines can be largely overcome by the use of a circulating system, as mentioned by the writer under "Grouting Equipment." Of course this does not prevent settlement of cement in the grout in the hole itself; but it does bring well-mixed fresh grout much closer to the point of use than a single direct pipe line. Where a large enough hole is used, a header can be arranged with the grout pipe feeder extending to the bottom of the hole through the larger casing. Grout is fed through this pipe and flows upward and on through a connection to the casing back to the pump sump. This method permits washing out the entire hole where and when desirable or necessary. This system was used at Chickamauga Dam and was mentioned by James S. Lewis, Jr.,[37] Assoc. M. Am. Soc. C. E. Costs are not yet available on the Chickamauga Dam, pending the completion of the final report on the project.

At Chickamauga Dam, in the area downstream from both embankments, wells were installed for observing the water table. Filling of the reservoir was started about January 16, 1940, and normal pool elevation was reached about April 1, 1940. To date (November 1, 1940) no fluctuation of the ground water has been caused by the reservoir.

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[37] "Large Core Drills Aid Construction at Chickamauga Dam," by James S. Lewis, Jr., *Transactions* Am. Soc. C. E., Vol. 105 (1940), Fig. 6, p. 857.

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PERMISSIBLE COMPOSITION AND CONCENTRATION OF IRRIGATION WATER

By W. P. Kelley,[1] Esq.

With Discussion by Messrs. Carl S. Scofield, Walter W. Weir, Robert S. Stockton, Leon D. Batchelor, and E. B. Debler.

Synopsis

The limit of permissible salt content of irrigation water is greatly influenced by variables inherent in the soil, the climatic conditions, and the kind of crops grown. It is of the greatest importance to apply saline irrigation water in quantities in excess of the crop requirements, in order that some leaching of the root zone will take place. Therefore, the maintenance of good drainage condition in the soil is very important. Salts, whether native to the soil or applied in the irrigation water, cannot be removed effectively unless water can percolate through the soil, and this can never be accomplished adequately where the ground water is near the surface.

From the standpoint of irrigation supplies, there are two considerations of outstanding importance to irrigated agriculture, namely—(1) the physical availability of the irrigation supply, its cost of delivery, and abundance; and (2) the so-called quality of the water. The word "quality" as applied to irrigation water is not well chosen because it may refer either to the "ratio of dissolved constituents" or to "total concentration." The ratio of dissolved components does not have a fixed value in determining suitability for irrigation, because the influence of this ratio is affected considerably by the total concentration. Neither is the total content of salines highly determinative, because, within certain ranges of concentration, the kind of salines present has fully as much (perhaps more) influence than total concentration. Therefore, an irrigation water may be unsuitable because the kind of its components or its total concentration is unfavorable, or for both reasons. Hence the expression "composition and concentration" is preferable to the word "quality" as applied to irrigation waters.

Note.—Published in April, 1940, *Proceedings*.

[1] Prof. of Soil Chemistry, Univ. of California, Berkeley, Calif.

Early in history it must have been realized that the salinity of irrigation water is a matter of some importance. Otherwise, it would be difficult to account for the fact that irrigation supplies have frequently been developed at relatively great expense when other supplies were near at hand—as, for example, along the ocean front or near saline lakes. However, recognition of the fact that comparatively low concentrations of salines may also be important is more recent. In fact, as experience has accumulated and scientific investigations have progressed, the accepted limits of salinity have tended to diminish progressively. In many instances, water which at one time was thought to be entirely safe for irrigation has either been abandoned or is under suspicion.

The late E. W. Hilgard,[2] M. Am. Soc. C. E., did the pioneer work in formulating knowledge on this subject, and laid the foundation for most of what is known about it today. He realized at the very beginning of his important investigations on irrigated agriculture that the farmer must reckon with the dissolved salines of irrigation water. Soon after taking up his work in California, in 1875, Hilgard began the analysis of irrigation waters, and from that date to the present time irrigation supplies have been freely analyzed for interested farmers in California. It is well to recall the pioneer work of Hilgard and to bear in mind that practically everything that has been done on this subject since his time has been chiefly in the nature of refinements of the fundamental ideas which he developed and advocated through a long period of highly productive research. Not always were Hilgard's ideas entirely palatable to farmers or promoters, but rarely has time failed to show their essential soundness.

Although Hilgard was a true pioneer in this field, others soon caught his vision. R. H. Forbes[3] and his successors in Arizona, and F. S. Harris[4] and his collaborators in Utah, have long recognized the importance of dissolved salines in irrigation water, and the same is true of certain members of the U. S. Department of Agriculture, notably T. H. Means[5] and C. S. Scofield.[6] Therefore the subject is far from new.

During quite recent years the subject of composition and concentration of irrigation water has come to the forefront. In fact, certain workers have concluded that a very high percentage of irrigated land in the United States (estimated at more than one half) is threatened with becoming excessively saline, owing to the salt content of the irrigation water. The writer does not subscribe entirely to this apprehension; the estimate is much too high, and unnecessary alarm has been aroused in certain sections.

It is perfectly true that large areas of irrigated soils have become excessively saline since the time irrigation was first introduced. For example, it is estimated that several hundred thousand acres of the more than six million acres have been thus injured in the State of California, and areas of substantial size have been affected in practically every other irrigated state. However, the evidence is not convincing that the salts that have become concentrated in the soil have originated chiefly in the irrigation water that was applied. It is true that there are certain localities in which the injury has been clearly produced by the salines of the water applied, and other areas where severe injury from this cause is more than a reasonable prospect, either immediate or remote; but the greater part of the injury thus far has come about through the capillary rise of salts from the deep subsoil, made possible by a high water table. Imperfect drainage conditions threaten irrigated agriculture far more seriously than salinity of irrigation supply.

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[2] Annual Repts., Agricultural Experiment Station, Univ. of California, 1878–1902.

[3] *Bulletin No. 44*, Agricultural Experiment Station, Univ. of Arizona, 1902.

[4] *Bulletin No. 139*, Agricultural Experiment Station, Univ. of Utah, 1919.

[5] *Circular No. 10*, U. S. Dept. of Agriculture, 1903.

[6] *Bulletin No. 40*, Dept. of Public Works, State of California, 1933.

Nevertheless, salinity of irrigation water is a matter of some importance. Unfortunately, however, it is difficult to name the exact limits of permissible concentration of salines. The principal reason why this is not feasible at present is that too many variables are involved, the range of which is very great. At best the irrigation engineer can set up only tentative limits, and at present these must be based mostly on practical experience in the localities in question. This will probably always have to be the case, because permissibility can never be a fixed quantity applicable any and everywhere, but a relative matter depending on soil, crop, and climate. A reasonably safe water for one soil or crop may be very questionable for another.

To illustrate: A certain water, which has been used in a few orange orchards near Riverside, Calif., has produced severe defoliation and other injury to the citrus trees. However, it was found that alfalfa can be grown quite successfully with the same water. A second example is that of a 10-acre orange orchard near Lindsay, Calif. Some years ago, after having been irrigated successfully with non-saline water for a period of about twelve years, this orchard was irrigated with water from a well that contained a rather high concentration of chlorine. In one section the citrus trees became almost completely defoliated after this well water had been used for only two or three years, whereas in another part of the same grove the trees were scarcely affected at all. The only known difference between the two parts was in the soil. Where the injury was severe the soil (San Joaquin sandy loam) was shallow, being underlaid with an impervious hardpan about 2.5 ft from the surface. On the other hand, where the injury was only slight, the soil (Hanford fine sandy loam) was porous and deep, no hardpan being found. These two instances were spectacular, but in many other cases the effects have been much less pronounced and are often difficult to determine.

In a third locality, lemon trees were found to be injured, decidedly, by water containing somewhat less than 1 ppm of boron, whereas beans and alfalfa grown as intercrops showed no detectable injury. Differences in the response to boron of different kinds of crops are sometimes great indeed, as Eaton has clearly shown in controlled sand cultures at Riverside.

Many other instances could be cited, but these will suffice to indicate that the character of the soil to which the water is applied and the kind of crop that is grown have very great influence on the question of permissible salinity of irrigation water. On a given soil, water that is entirely safe for one crop may produce serious injury for another, and on soils of wide dissimilarity severe injury may be produced on the one and practically none on the other.

The soil and crop are by no means the only important variables. Climate is probably just as important. The rainfall has a tremendous influence. For example, a lemon grove near La Habra, Calif., is irrigated with a rather saline water. During each of two springs following winters of heavy rainfall, the soil solution has been found to be no more concentrated than that of groves irrigated with good water. However, in the autumn, before the rainy season, there was considerable difference. Therefore, rain may leach out whatever salines accumulated during the previous irrigation season. In such cases the irrigation engineer must reckon chiefly with the concentration that accrues from a single year of irrigation; but where the rainfall is too light to effect much leaching, the soil solution, after a few years of irrigation, may become considerably more concentrated than it was at the close of the first year of irrigation.

However, where saline water is being applied under light rainfall, it should not be inferred, necessarily, that the concentration of the soil solution will continue to increase indefinitely. If the soil is reasonably porous the irrigation water itself may effect a substantial degree of leaching and thus transport, down below plant roots, some of the salines that have been introduced previously. Obviously, the permeability of the soil is a large factor in all such cases. With soil that is underlaid with an impermeable hardpan near the surface, manifestly little leaching can occur, and in such case practically all the salines applied must remain in the soil; and such soil will ultimately become highly charged with salts irrespective of the rainfall. If the subsoil has low permeability the tendency will be in the same direction, although perhaps less aggravated. Since soils vary enormously in their permeability, variations in the permissible limits of salinity must be considerable for this one reason alone.

Finally, the effects of salines appear to be influenced by another variable—the atmospheric conditions under which the crop is grown. If the rate of evaporation is rapid, obviously water must be applied more frequently than in relatively cool, moist climates. In such cases (other factors being equal) the total quantity of salines actually applied in a given period of time will be influenced by the climate. However, there is some indication that the effect on the crop itself is influenced by the atmospheric conditions; but this type of effect is in need of further investigation.

At any rate, one can state, safely, that variations in soil type, kind of crop grown, and rainfall are all potent in influencing the effects and acceptability of saline irrigation water; and since the complex differences in soil, crop, and climate present an intricate series of variations, it is scarcely probable that generally applicable limits of permissible salinity can ever be established. Each locality is largely a problem unto itself. Also the method of application has some influence. Leaching certainly will tend to be most uniform and effective if the water is applied by the flooding or sprinkling system; but, of course, the quantity of water applied has most influence. If applied by the furrow method, the lateral movement of moisture, in some instances, causes great lateral inequalities in the distribution of soluble salts, especially in the top soil. In certain cases soil scientists have found a ten-fold greater concentration between irrigation furrows than directly below the furrows. This type of inequality of distribution sometimes extends to a depth of 3 ft or more.

That the salts applied in the irrigation water may penetrate to very great depths, and therefore not remain permanently in the root zone, has been shown clearly by the results of an investigation on an orange orchard near Riverside. This orchard had been irrigated with fairly saline water since 1910. The annual rainfall is comparatively light, being about 11 in. The irrigation water was applied by the furrow method and the quantity per irrigation was similar to that of the general commercial orchard practice of that locality—about 30 acre-in. per yr. Certainly the farmer did not intentionally apply excessive quantities of water. The soil and subsoil of this orchard are sandy loams to great depth, and free from hardpan. The ground-water level is at least 150 ft below the surface.

When this soil was sampled to a depth of approximately 40 ft, it was found that the concentration of salines that originated in the irrigation supply, although somewhat variable in the samples from different sample holes, nevertheless was remarkably constant throughout the entire depth. It is true that there were zigzags in the curve of distribution, but the trend was toward a constant composition throughout the depth sampled. It is interesting to note that, on the average, the concentration of the soil solution, computed for a moisture content corresponding to that of field capacity of the soil, was somewhat less than three times the concentration of the water applied. The net influence of evaporation and transpiration, on the one hand, which tended to increase the concentration of the soil solution, and the downward movement and dilution of salines by rains and by irrigation water, on the other hand, which tended to reduce the concentration, jointly produced an increase in the concentration of the soil solution of only approximately three-fold that of the water applied. The fact that the concentration of the salts tended to be constant throughout the depth of 40 ft strongly indicates that this particular irrigation water could be applied to this soil indefinitely without producing a soil-solution concentration much in excess of three times that of the water applied.

One of the critical points in this entire question is not so much the salinity of water that is applied as the resulting effect that will be produced on the soil solution. The effect on the soil solution produced by a given saline water is highly variable. The variation depends almost entirely upon the properties of the soil to which it is applied, the rainfall, the frequence and quantity applied, and the method of application. As indicated previously, the method of application may be of considerable importance, particularly if there is a plow-sole (a firm, rather impervious, layer created by too frequent plowing at the same depth), or if the soil permits only slow penetration of water. In these cases the lateral movement of water may result in the development of much high concentration of salts between irrigation furrows. In the aforementioned grove, although the soil is quite porous and practically free from plow-sole, nevertheless considerable difference was found in the twenty individual samples that were drawn from each of two different parts of this grove. In one part of the orchard the injury to the orange trees was much more severe than in other places, and here the concentration of soil solution was considerably higher, particularly in the upper part of the soil. In another grove irrigated with this same water, the orange trees were literally killed by the salts of this water, whereas in other parts of the same orchard the trees (although somewhat injured) are still bearing fruit after the continued use of this water for more than thirty years. Undoubtedly, slight differences in soil, although difficult to measure, have a profound influence on the movement of salines and therefore on the effects that will be produced on crops.

It seems scarcely necessary to reiterate that results such as the foregoing may not be obtained on all soils and under all conditions. However, this much can be stated definitely: If the water penetrates below the depth of the crop roots, it is certain that the salts will be displaced downward to some extent after each irrigation, and will finally make contact with the ground water; or a perched water table will develop. Furthermore, this will be the result even though the soil is relatively heavy, provided penetration can take place at all. If the movement of water through the soil is slow, a longer period of time after the water is applied will be required to bring about equilibrium of distribution. In the aforementioned orchard approximately constant moisture conditions were found from the surface of the soil down to a depth of 40 ft three days after irrigation. This is further evidence of the freedom of movement of moisture through this soil.

On the other hand, if the amount of water applied is no more than enough to wet the soil of the root zone, then the salts applied must necessarily accumulate in the root zone and ultimately will produce an injurious concentration even though the salt content of the water is low. Therefore, it is extremely important, where saline water is used, to apply more water than is necessary merely to wet the root zone. A certain degree of leaching must be provided by applying water in excess of the water requirements of the crop; otherwise the root zone will inevitably become highly charged with salts. Obviously, the more saline the water applied, the sooner will injurious concentration result in the soil solution, but in such cases water should not be applied sparingly. The more saline the water is, the more should be applied in order that leaching may occur every time water is applied.

Irrespective of where the water is applied, or the kind of crop grown, boron is one of the most toxic constituents found in the irrigation waters of California. It is doubtful whether water containing more than 0.5 ppm of boron can be applied to orchard soils continuously without ultimately producing some injury. The on-set of the injury will appear after varying periods of time, depending on the soil, frequence of irrigation, and to a lesser extent on the species of fruit trees grown, the fertilization, and general management of the orchard.

Among the more common constituents of irrigation water, chlorine is perhaps the most to be feared, but different kinds of crops vary widely in their response to chlorine. With citrus trees grown in California the water should usually not contain more than 150 to 200 ppm chlorine. Soluble carbonate is more toxic than chlorides, but it is much less widely present in irrigation waters. Sulfates are somewhat less toxic than chlorides.

In conclusion, a word is in order about sodium. There certainly is some importance to the ratio of sodium to calcium in irrigation water. This is true, because that ratio has an important bearing on the absorption of sodium by the soil. If the amount of absorbed sodium in the soil becomes substantial, the physical properties of the soil tend to become affected adversely; and if the proportion becomes extreme, the permeability of the soil will be impaired seriously. Present information indicates that very little sodium will be absorbed by the soil unless the ratio of sodium to calcium present in the soil solution is greater than 2 : 1. However, it is important to realize that the ratio of sodium to calcium in the actual soil solution may become substantially greater than it is in the water applied. This is due to the fact that calcium is absorbed by crops, and is therefore removed from the soil to a much greater extent than sodium; or it is precipitated as insoluble carbonate, which produces the same type of effect in this ratio.

Perhaps enough has been said to indicate the general trend of present-day knowledge and to give some idea of the great complexity of this subject. It is gratifying to find that the engineer is taking an interest in this highly complicated subject, in its chemical, physical, and physiological aspects. Every one is working toward the same goal, and each should profit by the experiences of those working in related lines.

DISCUSSION

Carl S. Scofield,[7] **Esq.** (by letter).—Engineers would like to have a rule to establish the composition of irrigation water. The author recognizes this fact, and the major objective of his paper is to show the difficulty or impossibility of drafting such a rule. The crux of the matter is put into one sentence: "One of the critical points in this entire question is not so much the salinity of water that is applied as the resulting effect that will be produced on the soil solution." In other words, it is the composition and concentration of the soil solution rather than of the irrigation water that determine the productivity of a field or of an area within a field.

It is almost universally true that the soil solution is more concentrated than the irrigation water by which it is replenished. This follows because the soil solution loses water by evaporation and the dissolved constituents do not evaporate, and also because, in general, plants absorb from the soil solution a higher proportion of water than of dissolved constituents. Thus, both evaporation and plant absorption tend to concentrate the soil solution. The input of rain water dilutes the soil solution; and root-zone leaching, either by rain or by copious irrigation, displaces the soil solution within that zone. Thus, the concentration of the soil solution may range up to manyfold that of the irrigation water, and the ratios of the two concentrations may be very different at any time in different parts of the same field or at different times in the same location.

Even in respect to the concentration of the soil solution the criterion as to the upper permissible limit must be a matter of definition. For a given plant in a given climatic environment a certain constituent concentration of the soil solution (for example, 500 ppm of chloride) may cause a measurable depression of growth—say a reduction to 80% of normal. For the same plant and environment increasing concentrations cause further depression of growth until at possibly 3,000 ppm the growth may be only 10% of normal. With such a wide range between the concentration that causes measurable injury and one that would kill the plant, there is room for a difference of opinion as to the permissible concentration of the soil solution for a given plant in a given environment. It may be added that among crop plants there are wide differences of tolerance to any salt constituent. To produce equivalent growth depression by the same constituent in two different crop plants may require as much as a five-fold difference in solution concentration. Furthermore, the salt constituents differ greatly in their effect. For example, with some plants boron is at least 100 times as toxic as chloride and there is reason for believing that with some plants chloride is at least twice as toxic as sulfate.

Because of these facts: (*a*) That there is no consistent relationship between the composition and concentration of irrigation water and that of the soil solution; (*b*) that crop plants differ widely in their reactions to the same salt constituent; (*c*) that climatic conditions influence plant reactions to salt constituents; and (*d*) that there is no generally accepted criterion as to what degree of crop injury is "permissible," Professor Kelley has refrained from giving a definitive list of permissible concentrations. In this the writer believes the author has been well advised.

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[7] Agriculturist, U. S. Bureau of Plant Industry, Washington, D. C.

It would be possible to formulate an acceptable list of constituent concentrations for irrigation water below which, for any given soil condition and climatic environment, the danger of salinity injury to crops would be so remote as to be negligible. It would also be possible to make another list of constituent concentrations for irrigation water above which the danger of salinity injury would be almost certain. Between these two lists would lie a broad zone within which "permissibility" would be conditioned by soil type, subsoil drainage, kinds of crops, climatic conditions, and adequacy of water supply.

Engineers would do well to read carefully the fifth from the last paragraph of the paper. In this paragraph Professor Kelley discusses the need of copious irrigation and root-zone leaching where saline irrigation water is used. The thesis thus presented is one that the writer heartily supports.

WALTER W. WEIR,[8] ASSOC. M. AM. SOC. C. E. (by letter).—The engineer would like to know how much salt may be contained in irrigation water and still leave it suitable for irrigation purposes. Mr. Kelley has very carefully avoided a direct answer to this question, notwithstanding the fact that he is probably as well-informed regarding the chemical analysis of irrigation waters as any one in the West. It is a relatively simple matter to determine the amount of soluble salts present in any given sample of water, and with a knowledge of the environmental and cropping conditions—as well as the physical and chemical composition of the soil upon which the water is to be used—it is possible, with reasonable accuracy, to predict with what safety the water may be used. Generalizations are dangerous, however. For example, the waters used in the Salt River and Imperial valleys are relatively high in soluble salts, particularly chlorin and sulfates, and yet where drainage conditions are good may be used with safety in these localities or on soils such as those of the Imperial and Salt River valleys which are high in calcium. Irrigation waters derived from the Kern, Kings, and San Joaquin rivers, and many of the well waters used in the vicinity of Fresno, Calif., are exceptionally low in total dissolved salts, but, because of the nature of those salts which are present, and the particular soils on which they are used, they have resulted in some injury. In the soils of this area the ratio of calcium to sodium is low, and the continued use of these waters would eventually render these soils unfit for agriculture except for the fact that most of the soils are pervious and drainage is good and water is plentiful. Wherever, on the east side of the San Joaquin Valley, drainage is poor and high water tables have developed, the sodium carbonate in the irrigation water is beginning to show its effect.

Mr. Kelley has shown very clearly that poor drainage, whatever may be its cause, is likely to be a more important factor than the salt content of the irrigation supply. The other important point that he emphasizes is the necessity of using more water for irrigation than is required for plant growth; in other words, leaching beyond the root zone is essential to the continued success of irrigation farming even if the salt content of the water used is low. It must be realized, however, that the use of excessive quantities of irrigation water may leach some desirable salts or soluble nutrients from the soil, as well as undesirable salts, and that the dangers of a high water table and poor drainage are increased as the use of water increases. If the soils to be irrigated are sufficiently pervious to provide good drainage and leaching below the root zone, and sufficient water is used to keep the upper soils leached of excessive accumulations of salts, it is possible to continue to irrigate with waters which contain rather high concentrations of alkali salts. On the other hand, if drainage is poor or if only limited quantities of water are available, so that leaching rarely occurs, even small quantities of salts in the irrigation water may eventually become troublesome. Therefore, it is obviously impossible, from present knowledge of saline or alkali waters, to make a definite statement as to the amount of alkali salt that water may contain and still be safe for irrigation purposes.

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[8] Drainage Engr., Univ. of California, Berkeley, Calif.

ROBERT S. STOCKTON,[9] M. AM. SOC. C. E. (by letter).—It is desirable that engineers and the courts should take cognizance of the fact that long continued use of water for irrigation on certain lands has a tendency to load the soil with salts deleterious to plant growth.

The accumulation of salts may be fairly rapid as in the case of alkaline unproductive areas that have developed within most irrigation systems within a few years after water is applied, due mostly to rising water tables and the concentration of salts already in the soil, and only to a slight extent due to salts in the water applied.

As the author states, the quantity of salts in water used for irrigation varies widely; hence the effects, which depend also on the soil, climate, and other factors, may occur in a relatively few years, or generations of users may spread the water before great damage occurs. This slow filling of the soil with salts was the probable cause of the abandonment of many or, perhaps, all of the ancient irrigation systems whose traces may now be found in many parts of the world.

The present knowledge of these facts is of importance in designing new irrigation systems, in laying down the policy of operation and maintenance, and establishing methods of properly using the water available, as well as in protecting values under established systems.

Some lands will no doubt become unproductive in spite of any practical method of preventing or curing the trouble, but, since there is so much more land in need of irrigation than there is water to serve it, there is usually a good chance that the water can be used on new lands with greater precautions and better methods. Since these changes usually occur very slowly, there is a chance to distribute the capital loss over a period of years.

The author emphasizes the use of excess water to carry away excess salts, but this must not be taken as permission to countenance waste of water without the most careful agreement by all authorities that such waste is justified. Unless very carefully supervised, the use of excess water may cause much more alkali trouble by raising water tables and leaching the soils than any other factor. In fact, the great fight of irrigation managers and conservationists is to prevent waste of water with its attending damages and reduction of the area that can be served from any given source.

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[9] Cons. Engr., Irrig. and Land Development, Craigantler Ranch, Thompson Falls, Mont.

One may say that the water shall be applied so carefully and skilfully that there shall be, as far as practicable, no upward movement of applied water. This is insured, in large measure, by cover crops, such as grassland meadows, and by prompt and thorough cultivation of all cropped land after each irrigation, so as to establish a mulch and thus reduce evaporation of water from the soil.

The great practical difficulty in the field is in controlling and directing the water users, who are in most cases struggling to make a living and lack time and help to attend to irrigation properly; they feel that they must do things the easiest way and are often wedded to wasteful and unscientific methods. It is most essential, therefore, that court decisions and irrigation engineers and managers shall do all that can be well done to educate the public and enforce those methods that shall best protect land values and production for future generations, as well as for the more or less immediate present.

The probability of the concentration of alkaline salts in soils under irrigation justifies a much greater expenditure for deep drainage than was formerly thought necessary. Many irrigation systems have had to increase the drainage system from time to time, and we may expect, in a large number of cases, that the drainage system will ultimately be nearly as extensive as the water distribution system. These considerations may help to construct, when feasible, the drainage system before the lands are rendered unproductive and thus save expensive reclamation or loss of land. This necessity for drainage, of course, shows that there is in fact an excess of water used, and the problem is to provide for it and at the same time use water so that there will be as small a waste as practicable, but still providing for a slight downward percolation.

In conclusion it may be said that irrigation engineers should never lose sight of the losses in soil fertility due to irrigation that puts water below the root zone; and further, that if soils are kept rich in nitrogen and humus, very much less water is necessary to be supplied by irrigation, and evil effects are minimized.

Leon D. Batchelor,[10] **Esq.** (by letter).—There has been much confused thinking on the subject of permissible composition of irrigation water, which could be materially clarified by a careful consideration of this article. The author is to be commended for his summarization of his own research and that of others on this important subject.

Even greater emphasis might properly have been placed by the author on the rôle of the native salts in the soil and subsoil, prior to irrigation, in causing subsequent injury to crops. The movement of such salts to the root zone of crops, by capillary rise of the soil moisture from a perched water table, is recognized by Professor Kelley, as it often has been by various students of this subject. Unfortunately, precise data relative to the native salt content of the soil do not exist in most irrigated sections. A notable exception to this is the Imperial Valley, in California, in which a survey was made prior to any irrigation, and salt determinations were made of the surface soil, lower root zone depths, and, in two cases, the deep subsoil to a depth of 22 ft. The results of this investigation were published[11] in 1902, long before it was generally recognized that the salt and drainage problem was to be a serious consideration in the Imperial Valley. Tests of rate of percolation on various soil types were also reported in this bulletin. These tests, together with the salt determinations which showed enormous amounts of native salt in the subsoil, implied, in advance, much of the difficulty which actually has been encountered in this area. These early studies also enable one to evaluate, to some extent at least, the part played by the irrigation water on one hand and by the native salts of the soil on the other hand in bringing about a salt problem in the area. Unfortunately, such fundamental preliminary data concerning irrigated areas are not commonly available.

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[10] Director, Univ. of California Citrus Experiment Station, Riverside, Calif.

In the explanation of the effect of ordinary irrigation in leaching the salts below the root zone of plants, the sound procedure, in general, is emphasized that the more saline the water is the more water should be applied in order that leaching may occur every time water is applied. It is the belief of the writer that this could be overdone beyond the point of accomplishing the desired end of avoiding the accumulation of salt in the root zone. First, it seems doubtful to the writer whether irrigation water is ordinarily applied to most crops in such quantities that the amount of water is no more than enough merely to wet the soil of the root zone. The opinion is held that in the vast majority of instances, in the case of annual crops and shallow-rooted tree crops, the casual irrigation causes a notable amount of leaching. As pointed out by data prepared by the author, the rains are an important factor in leaching the salts out of irrigated lands. If leaching is promoted by the use of extra irrigation water in addition to normal rainfall, it should be done only when precise data show its need. Otherwise, an unnecessary loss of nitrates from the soil may be the result of an uncalled for attempt to reduce the salt concentrations.

E. B. DEBLER,[12] M. AM. SOC. C. E. (by letter).—A timely contribution on a subject of increasing importance in the field of irrigation has been presented by Professor Kelley. The relationships of the various factors, which determine the permissible composition and concentration of water that may be used safely for irrigation, have been illustrated clearly, stressing the fact that saline water is not alone in contributing to salt injury of soils. Drainage has long been recognized as playing a major rôle in determining the extent to which salt accumulation takes place. The other factors discussed, such as type of crop, climate, and method of applying water, simply serve to fix the quantity of water that must be applied to maintain a concentration in the soil solution within a range favorable for plant growth.

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[11] Discussion by E. W. Hilgard and G. W. Shaw of "Lands of the Colorado Delta in the Salton Basin," by F. J. Snow, *Bulletin No. 140*, Univ. of California, 1902.

[12] Hydr. Engr., U. S. Bureau of Reclamation, Denver, Colo.

Rightfully, the author emphasizes the fact that in order to maintain a favorable concentration of the soil solution, the movement of dissolved salts must be maintained in a downward direction. This is not possible unless adequate drainage exists and sufficient water is applied in excess of crop use and evaporation loss to provide percolation through the soil profile. Cases are on record in which water containing 4,000 ppm and more of dissolved solids has been used continuously without any apparent deleterious effect on plant growth. However, this has been accomplished through copious irrigations on permeable soils where well-developed drainage systems exist.

A common error among irrigators has been the use of insufficient water where the salinity content was high. This practice prevented leaching and consequently resulted in excessive accumulation. Use of sufficient water to insure thorough leaching of the root zone is not interpreted as being a wasteful practice, but rather a necessity for permanence in successful irrigated agriculture.

Professor Kelley is to be commended on his presentation. It brings to the field of civil engineering information that is basic in the success of irrigation enterprises.

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Paper No. 2115

DESIGN OF HINGES AND ARTICULATIONS IN REINFORCED CONCRETE STRUCTURES

By George C. Ernst,[1] Assoc. M. Am. Soc. C. E.

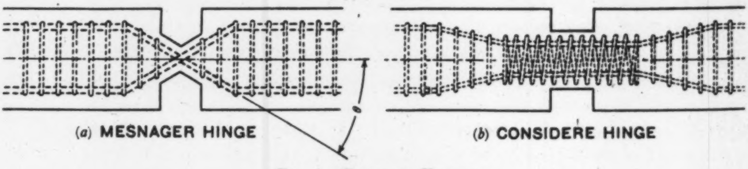
With Discussion by Messrs. A. A. Eremin, and George C. Ernst.

Synopsis

Information on the design of reinforced concrete articulations is not readily available for convenient review and use by the designer. The purpose of this paper is (*a*) to provide a better understanding of the action of such joints when subjected to thrust, shear, and rotation, by a review of the most important features of past tests, and (*b*) to submit methods of design that will yield appropriate sections.

Introduction

The desire on the part of many designing engineers to provide a relative freedom of rotation at predetermined points in reinforced concrete structures has brought into use the so-called semi-articulated, or reinforced concrete, hinge illustrated in Fig. 1. They may be permanent or temporary depending upon whether the structure is to be statically determinate throughout its life or only during the period of rapid volumetric change caused by plastic flow, shrinkage, rib-shortening, and the early temperature changes. In either case, it has been found expedient to make the hinge of reinforced concrete, as an integral part of the structure.



(*a*) MESNAGER HINGE

(*b*) CONSIDERE HINGE

Fig. 1.—Types of Hinges

Because design methods based upon theoretical analysis, supported by laboratory tests, have not been available, hinges have been constructed fre-

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Note.—Published in April, 1940, *Proceedings*.

[1] Asst. Prof., Civ. Eng., Univ. of Maryland, College Park, Md.

quently which were not free to rotate properly. Furthermore, reinforced concrete hinges often have been specified because of their novelty rather than their need. Their use is frequently questionable when a high shear is known to exist unless they are detailed with the center line of the hinge virtually collinear with the line of thrust. A review of test results supplemented by design methods and charts for the practical use of the two principal types (see Fig. 1) should supply a long-felt need.

Notation.—The letter symbols used in this paper are defined where they are first introduced and assembled for reference in Appendix I.

TESTS

Mesnager Hinges.—The original tests of A. Mesnager (1)[2] consisted of two specimens subjected to thrust and rotation. They showed, definitely, the feasibility of such construction. The two hinges consisted of sixteen bars 0.79 in. in diameter making an angle of 22° 30′ with the center line (yield point = 42,600 lb per sq in.). The hinges were encased in mortar, and the slenderness ratio $\frac{l}{r}$ of the bar was equal to 21. One specimen was rotated 0.02 radian by means of a steel wedge at one end and then loaded to failure in the rotated position. The ultimate load was 323,000 lb. The other specimen received the same relative rotation (0.02 radian) by an application of wedges at each end. A load of 154,000 lb was applied and released twice prior to reversing the wedges and repeating the loading twice again. The mortar covering was then removed from the hinge and the procedure was repeated prior to loading to failure under a rotation of 0.02 radian. The ultimate load was 238,000 lb. Failure in both specimens resulted from insufficient lateral reinforcement in the end blocks near the hinge opening.

Tests upon seven hinges were reported in 1935 by D. E. Parsons, M. Am. Soc. C. E., and A. H. Stang (2) of the National Bureau of Standards. Three of the hinges contained six ⅝-in. bars with a slenderness ratio, $\frac{l}{r}$, of 32.4, and the remainder contained eight ½-in. bars with $\frac{l}{r}$ equal to 39.6. The yield strength of the ⅝-in. bars was 43,600 lb per sq in., and that of the ½-in. bars was 46,200 lb per sq in. Four of the specimens had mortar covering at the hinge and the bars of all specimens made an angle of 30° with the center line of the hinge. Lateral reinforcement consisting of ¼-in. stirrups at 3-in. centers was placed in an attempt to prevent cracking of the end blocks. All except two, however, developed such cracks. These two had eight ½-in. bars without mortar covering and one was subjected to a shearing load of 0.292 times the thrust. Four of the specimens which were subjected to direct thrust received total rotations amounting to values from 0.011 and 0.026 radians by means of wedges at each end. In general, the results indicated that the bare bars of the Mesnager hinge could develop 90% of their yield point in direct thrust under the conditions of the tests cited. The work of Messrs. Parsons and Stang was outstanding because for the first time the testing of the combined effect of thrust and shear was investigated and because they developed formulas for the internal stresses in the bare hinge bars and the transverse reinforcement in the end blocks. The tests of Mesnager and the Bureau of Standards were discussed thoroughly in 1935 by Ben Moreell, M. Am. Soc. C. E. (3), who recommended a rational design procedure at the same time.

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[2] **Numerals in parentheses, thus (1), refer to the corresponding number in the Bibliography, Appendix II.**

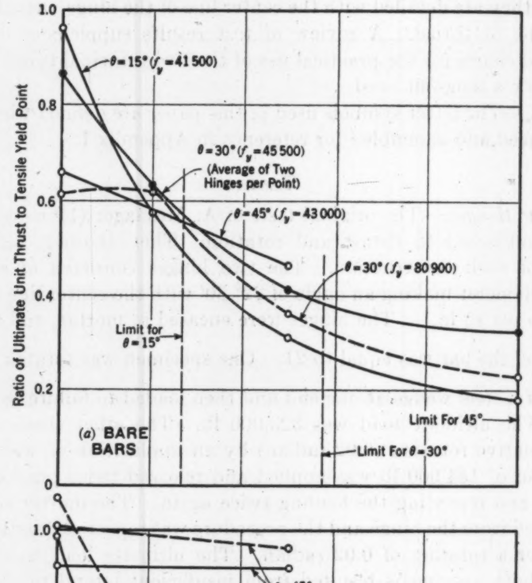

FIG. 2.—DEVELOPMENT OF THRUST ON MESNAGER HINGES (SLENDERNESS RATIO, $\frac{l}{r} = 40$; AND f_y = YIELD POINT)

In 1937 and 1938 tests were made upon Mesnager hinges at the Engineering Experiment Station of the University of Maryland, College Park, Md., for the purpose of checking, completely, the recommended method of design. These tests on 86 hinges (62 of which were reported in 1937 (4)) were the first to combine the effects of thrust, shear, and moment at ultimate and design loads. The investigation of the high shear-thrust ratios likely to be encountered in rigid frame design was also included for the first time in such tests. The Maryland tests, although on somewhat smaller specimens (¼, ⅜, and ½-in. bars), corroborated those of the Bureau of Standards and substantiated the design recommendations of Admiral Moreell.

Figs. 2 and 3, showing the essential data from the Maryland tests in which failure occurred in the hinge, lend emphasis to several new features which are of primary importance. In Fig. 2(*a*) the vertical broken lines are the limits for various values of θ; for example, at $\frac{V}{T} > 0.268$ the load passes outside the bars for $\theta = 15°$ (θ = the angle between the sloping bar and the center line of a Mesnager hinge, in degrees; and V and T = total shear and thrust components of the load P). In Fig. 2(*b*) the solid curves are based entirely upon test results. The broken lines in Fig. 2(*b*) are for an ultimate value of T with the hinge covered (as determined by test) and with the hinge bare (from Eq. 1, presented subsequently). In these tests the concrete strengths were: 2,580 lb per sq in. for $\theta = 30°$ and 1,910 lb per sq in. for $\theta = 15°$ and 45°. At about 30% of the ultimate load a rotation of approximately 0.01 radian, in both directions, was applied and released, the maximum eccentricity of load being 1.06 in. for $\theta = 30°$ and $\frac{V}{T} = 0$. In Eq. 1—the formula for maximum stress presented by Messrs. Parsons and Stang (2) and reviewed by Admiral Moreell (3)—the maximum stresses may be lowered by increasing the slenderness ratio since

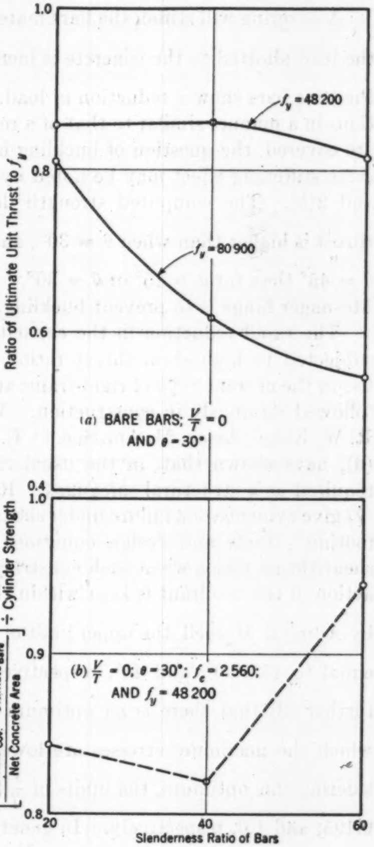

FIG. 3.—SLENDERNESS RATIO VERSUS DEVELOPMENT OF THRUST AND CYLINDER STRENGTH ON MESNAGER HINGES

the bars are thereby made more flexible. Fig. 3 shows the danger of increasing the value of the slenderness ratio beyond 40 for intermediate grades of steel or beyond 20 for steels having a higher yield point when the bars are not covered. The $\frac{l}{r}$-values of the Mesnager tests were 21 and those in the Parsons-Stang tests were 32.4 and 39.6.

A covering will stiffen the bars materially, as may be seen in Fig. 3(*b*), since the load allotted to the concrete is increased at $\frac{l}{r} = 60$ while at the same time the bare bars show a reduction of load. If it is recognized that the unit functions in a manner similar to that of a reinforced concrete column when the bars are covered, the question of buckling is seen to be of small importance. This same stiffening effect may be noted by comparing the 45° θ-lines in Figs. 2(*a*) and 2(*b*). The computed strength developed in the concrete under direct thrust is higher than when $\theta = 30°$; and, for the ratio $\frac{V}{T} = 0.5$, it is higher for $\theta = 45°$ than for $\theta = 15°$ or $\theta = 30°$. A major function of the covering in the Mesnager hinge is to prevent buckling.

The rapid reduction in the capacity of bare or covered bar hinges, when subjected to high shear-thrust ratios, should sound a warning against their use in the current type of rigid-frame structure unless the design of the hinge is followed rigorously in construction. W. M. Wilson, M. Am. Soc. C. E., and R. W. Kluge, Assoc. M. Am. Soc. C. E. (5), and A. J. Boase, M. Am. Soc. C. E. (6), have shown that, in the usual rigid frame bridge, such hinges are not required as a structural safeguard. Reports of Mesnager hinges in Germany (7) give evidence of a failure under shearing load, either in the hinge or adjoining footing. Tests and design equations point to $\theta = 45°$ for use under high shear-thrust ratios when such construction is deemed essential. If the line of action of the resultant is kept within the extreme hinge bars, as recommended by Admiral Moreell, the upper limit of $\frac{V}{T}$ is 0.268, 0.577, and 1.0 for values of θ equal to 15°, 30°, and 45°, respectively (see Fig. 2(*a*)). It has been shown further (3) that there is an optimum value of θ, for a given value of $\frac{V}{T}$, for which the maximum stresses are lower than for all other values of θ. Considering this optimum, the limits of $\frac{V}{T}$ for $\theta = 15°$, 30°, and 45° become 0.024, 0.195, and 1.0, respectively. In general, the curves of Fig. 2(*a*) indicate that it would be best not to exceed the $\frac{V}{T}$-ratio for which a given θ is the optimum value. The hinge unit may be sloped sufficiently to meet this requirement, if necessary.

For any values of θ and $\frac{V}{T}$, the concrete develops at least $0.8\left(1 - \frac{V}{T}\right)$ of the cylinder strength. A factor of safety of 2.5 would allow for $0.32 f_c'\left(1 - \frac{V}{T}\right)$ to be taken by the minimum concrete section in the design of hinges with mortar covering, although recommendations for design have previously neglected the covering. This assumes a straight-line reduction from 0.32 at $\frac{V}{T} = 0$ to 0 at $\frac{V}{T} = 1.0$ in Fig. 2(*b*), for all values of θ.

In addition to the foregoing, all tests have shown rotation to have a negligible effect upon the ultimate capacity of hinges with bare bars and a marked effect upon those with covered bars, although a substantial increase over bare bars is still apparent. The capacities of hinges subjected to rotations applied and released prior to the ultimate have not been compared with those of unrotated specimens. It is apparent from Fig. 2(*b*), however, that a substantial resistance still can be supplied by the concrete under the former conditions. The resistance to rotation of either bare or covered bars is negligible.

It has been shown also (1) (2) (4) that a sufficient quantity of transverse steel must be placed close to the hinge opening and wired or welded in intimate contact with the longitudinal steel in order to prevent splitting of the adjoining members by the transverse components from the bars. Eq. 4, given subsequently, is an approximate formula for the determination of the quantity and position of the transverse steel, proposed by Messrs. Parsons and Stang. It gives satisfactory values in comparison with results from Parsons-Stang tests when the effective length for tensile resistance along the axis of the hinge is taken at not more than eight times the diameter of the longitudinal bars. The tests at the University of Maryland were conducted on considerably shorter specimens than those of the Bureau of Standards and the end blocks did not provide adequate embedment for bond in the case of the bars that were ½ in. in diameter. However, the formula for transverse reinforcement provided reasonable values for all tests in which the embedment for bond was not less than 22 diameters. For hinges with covered bars, the total shear V is taken by the sloping bars, as has been demonstrated by T. D. Mylrea, M. Am. Soc. C. E. (8), in the case of wedge shaped beams, but the opposing transverse components in the bars from the direct total thrust should be taken care of by web reinforcement.

Considère Hinges.—The Considère hinge is essentially a short section of spiral column, its dimensions being such that the resistance to rotation is small compared to that of the main adjoining members (see Fig. 1(*b*)).

In 1931 a synopsis of the results from several tests made in 1928 upon Considère hinges was published by C. B. McCullough, M. Am. Soc. C. E., and E. S. Thayer (9). These tests indicated the suitability of the Considère hinge for heavy duty service but did not supply any information upon the resistance of such construction to combined shear, thrust, and moment. Messrs. McCullough and Thayer stated, however, that the Grants Pass arch bridge, in Oregon, in which the Considère hinge was utilized, showed a vertical crown settlement in close agreement with the computed value when the arch was swung, which would indicate little angular restraint resulting from the stiffness of this type of hinge. Successful use of the Considère hinge on the Fourth Street Viaduct in Los Angeles, Calif. (10), was also reported by Leon Blog.

Additional tests were made at the University of Maryland in 1938 (11) upon sixteen Considère hinges using two concrete strengths, three spiral percentages, and shear-thrust ratios of 0, 0.2, 0.5, and 1.0. These hinges were subjected to rotations of as much as 0.018 radian approximately at design loads and in some cases well in excess of design loads. Figs. 4 and 5 provide a condensation of the test results for study and comparison with Mesnager hinges. The sets of three numerals at each plotted point signify, respectively: The percentage of the ultimate load at which rotation was applied; the maximum change in rotation, in radians; and the maximum eccentricity of the thrust, in inches. The test specimens were 5 in. by 5 in. in cross section at the hinge, with a 4-in. hinge opening. The spiral reinforcement was a 10-gage steel rod bent to a 4-in. diameter at a 1-in. pitch ($p = 1.43\%$); and the longitudinal reinforcement comprised four $\frac{3}{8}$-in. round rods ($f_y = 48{,}200$ lb per sq in.).

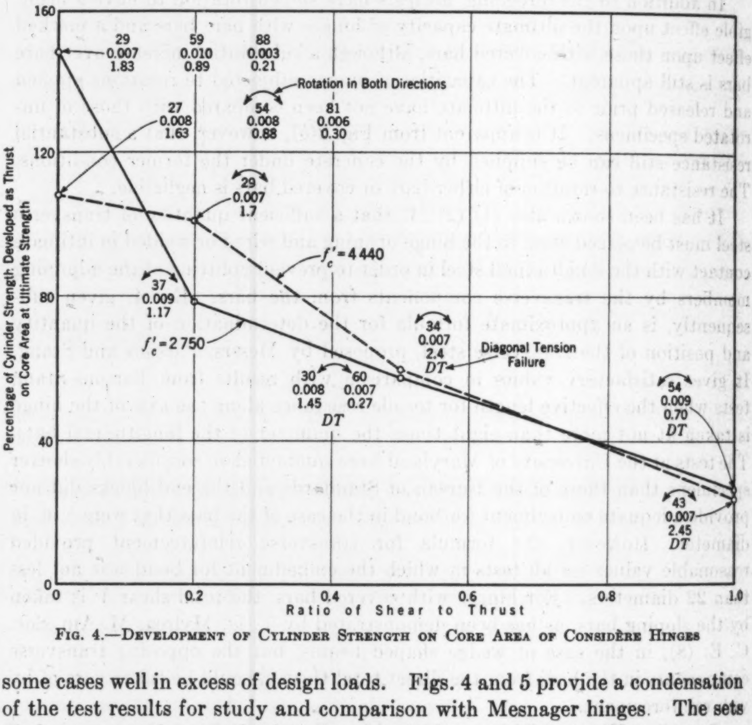

FIG. 4.—DEVELOPMENT OF CYLINDER STRENGTH ON CORE AREA OF CONSIDÈRE HINGES

The tests of Messrs. McCullough and Thayer show a development of 200% or more of the cylinder strength under direct thrust for spiral steel percentages of 3.25 and 6.50 (f_c' = ultimate strength of plain concrete = 6,000 to 6,400 at 60 to 90 days). This same effect is to be noted in Fig. 5 for hinges under short time loading and subjected to no rotation; but it is also extremely important to note that the spiral is ineffective in providing such a marked increase following rotations up to 0.011 radian. Rotations below about 30% of the ultimate load, for either grade of concrete, appear to be somewhat less damaging than those applied up to 88% of the ultimate load. In any case, however, it would be difficult to justify the use of more than 100% of the cylinder strength as the ultimate capacity for such construction under working conditions that do not include shear.

Fig. 4 shows essentially the same marked reduction in development of cylinder strength under shear as exists for the Mesnager hinge (Fig. 2(*b*)). The resistance to rotation as indicated by the maximum eccentricity of thrust of 2.45 in. would certainly be negligible in most cases.

It has been demonstrated (11) that the action of the Considère hinge is essentially that of a spiral column subjected to thrust and shear with a superimposed rotation that places the concrete in the plastic region on the high compression side. Such action suggests the limitation of deformation rather than stress during rotation, with preliminary proportioning for thrust the same as for a spiral column. The spiral may be utilized for protection against diagonal tension failures in the manner that is customary for members subjected to thrust and shear.

Considère Rollers.—Frequently, it is desirable to provide a connection which has no resistance to moment or shear. Although such construction generally has taken the form of a steel rocker or roller nest, it is entirely feasible to use reinforced concrete. The use of the Considère roller in reinforced concrete has been described by W. L. Scott (12) who also provides design formulas and procedure. Design methods for a variation of this type of unit are given by Sanford E. Thompson, M. Am. Soc. C. E., and the late Frederick W. Taylor, and E. Smulski (13).

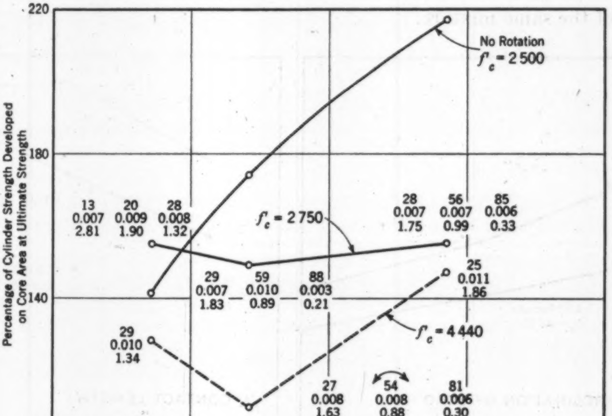

FIG. 5.—QUANTITY OF SPIRAL STEEL VERSUS DEVELOPMENT OF CYLINDER STRENGTH ON CORE AREA OF CONSIDÈRE HINGES $\left(\frac{V}{T} = 0\right)$

The test roller in Fig. 6 had no longitudinal reinforcement but a 10-gage spiral bent to 5-in. or 6-in. diameter, at a 1-in. pitch. In addition, grid stirrups of $\frac{3}{16}$-in. round rods were placed at the top and bottom of the spiral and in each bearing block so as to be approximately 0.5 in. from the contact faces. During casting, sheet steel forms were used to maintain the roller radius, as shown in Fig. 6(*a*). Twenty-two such rollers were tested with the Considère hinges described herein. Compressive strength was varied from 990 lb per sq in. to 4,750 lb per sq in. with roller diameters of 9 in., 12 in., 15 in., and 18 in., spiral steel percentages of 1.15 and 0.95, inclinations of load with center line of spiral of 0, 0.1, and 0.2 radians, and contact lengths of 7 in., 11 in., and 15 in. Deformations were measured to include those occurring in roller and concrete bearing blocks of the same mixture.

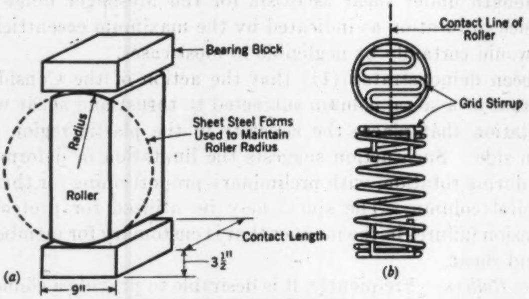
Bearing Block
Radius
Sheet Steel Forms Used to Maintain Roller Radius
Roller
Contact Length
$3\frac{1}{2}$"
9"
(a)
Contact Line of Roller
Grid Stirrup
(b)

FIG. 6.—CONSIDÈRE TEST ROLLER

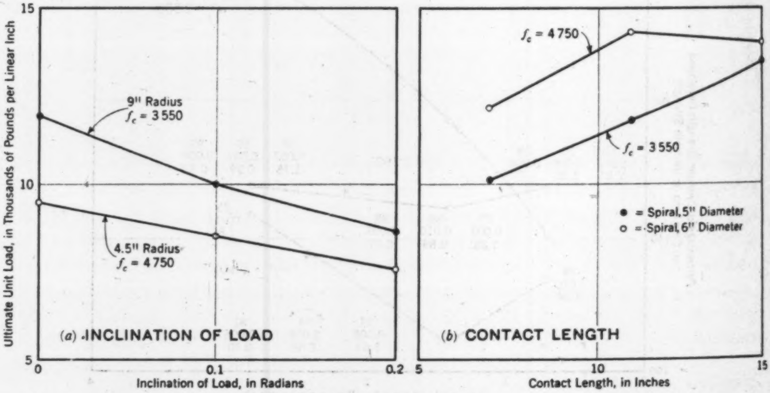

FIG. 7.—EFFECT OF INCLINATION AND CONTACT LENGTH ON UNIT LOAD; CONSIDÈRE ROLLERS

In these tests the upper limit of the straight-line part of the deformation curves occurred between 50% and 70% of the ultimate loads for concretes in excess of 2,000 lb per sq in. Failure was characterized by a vertical crack extending from one contact face to the other. The occurrence of this crack could be forecast by a rapid increase in the rate of deformation immediately before its appearance. Maximum sets in compression noted upon two applications and release of about 30% of the ultimate load were as follows:

Ultimate strengths of plain concrete, f_c', in pounds per square inch	Set, in inches
990	0.013
1,290	0.008
3,550	0.004
4,750	0.002

It may be noted further that an excessive inclination of the load may be serious, since the ultimate loads show a marked reduction with increases in the inclination (Fig. 7(*a*)). The use of rollers containing more than one spiral unit in order to increase the contact length shows some improvement over the single unit rollers.

Fig. 7(*b*) contains the results of tests on 12-in. rollers (radius = 6 in.). Rollers 11 in. long required 2 spiral units, and those 15 in. long required 3 spiral units. These units were overlapped 1 in. when the spiral diameter was 5 in. and 2 in. when the spiral diameter was 6 in.

DESIGN

Mesnager Hinges.—The design procedure for Mesnager hinges developed by Admiral Moreell (3) makes use of the following two equations for hinges with bare bars, presented by Messrs. Parsons and Stang (2):

$$f_s = \frac{T}{A}\left[\frac{K}{\cos\theta} \mp \frac{Q}{\sin\theta}\left(\frac{l}{r}\right)\right] \pm \frac{V}{A}\left[\frac{L}{\sin\theta} \pm \frac{R}{\cos\theta}\left(\frac{l}{r}\right)\right] + \frac{2\,E_s\,\phi}{\left(\frac{l}{r}\right)} \quad \ldots(1)$$

$$f_s' = \frac{T}{A\,(\cos\theta)} + \frac{V}{A\,(\sin\theta)} \quad \ldots\ldots(2)$$

in which: f_s and f_s' are the maximum, and the direct, compressive steel stress in a Mesnager hinge; A = the total bar area at the opening of the hinge; and

$$K = \frac{(l/r)^2}{(l/r)^2 + 12\tan^2\theta} \quad \ldots\ldots(3a)$$

$$Q = \frac{12\tan^2\theta}{\tan^2\theta + (l/r)^2} \quad \ldots\ldots(3b)$$

$$L = \frac{\tan^2\theta\,(l/r)^2}{(l/r)^2\tan^2\theta + 12} \quad \ldots\ldots(3c)$$

and

$$R = \frac{12}{12 + \tan^2\theta\,(l/r)^2} \quad \ldots\ldots(3d)$$

Eq. 1 (based upon fixed end conditions) yields maximum unit stress values as determined by adding the direct stresses and the superimposed bending stresses caused by T, V, and ϕ (the rotation of the hinge), whereas Eq. 2 provides a direct stress computed on the assumption of hinged ends at the face of the concrete for the bare bars, and neglecting ϕ. It has been recommended

(3) that the maximum stress computed by Eq. 1 be kept within the tensile yield point and that the direct stress computed by Eq. 2 should not exceed 30% of the tensile yield point (12,000 lb per sq in. for intermediate grade steel). The values computed by Eq. 1 agree well, and in general are slightly on the side of safety, with the results of tests shown in Fig. 2(*b*). The writer proposes that, for simplicity, design should be confined to the use of Eq. 1 by merely limiting the stress from Eq. 1, without rotational stress $\left(\frac{2\,E\,\phi\,r}{l}\right)$ to 30% of the tensile yield point instead of using Eq. 2. If this is done, the design chart shown in Fig. 8 can be used for all usual conditions in which the bars are contained in two planes.

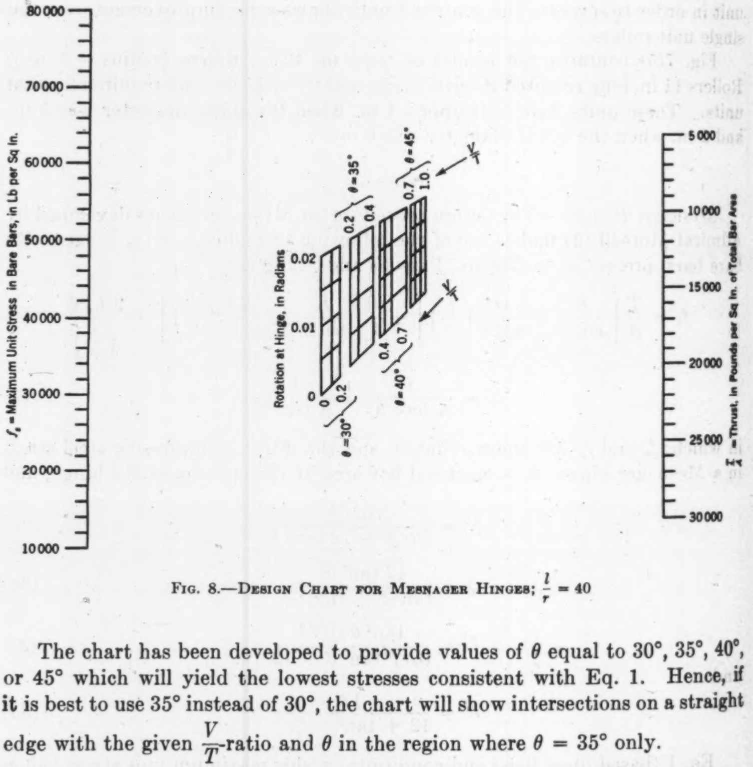

FIG. 8.—DESIGN CHART FOR MESNAGER HINGES; $\frac{l}{r} = 40$

The chart has been developed to provide values of θ equal to 30°, 35°, 40°, or 45° which will yield the lowest stresses consistent with Eq. 1. Hence, if it is best to use 35° instead of 30°, the chart will show intersections on a straight edge with the given $\frac{V}{T}$-ratio and θ in the region where $\theta = 35°$ only.

After determining the proper values of θ and $\frac{T}{A}$ for $\phi = 0$ and $f_s = 30\%$ of the specified yield point, the maximum stress should be checked against the tensile yield point by using the same diagram but including ϕ. If f_s exceeds the tensile yield point, $\frac{T}{A}$ must be redetermined by using the tensile yield point for f_s with the same values of $\frac{V}{T}$, ϕ, and θ.

If it is desired that an allowance be made for the concrete covering, the thrust taken by the concrete should be computed from $0.32\, f_c' \left(1 - \frac{V}{T}\right)$ times the minimum concrete section and this value deducted from the total thrust prior to computing the steel area from the $\frac{T}{A}$-ratio of the chart.

The percentage of transverse reinforcement may be computed by the following approximate equation given by Messrs. Parsons and Stang,

$$f_v = \frac{\frac{T}{2}\tan\theta + \frac{V\,a}{j\,d}}{0.005\,a\,b + A_v} \quad \text{.......................(4)}$$

in which: f_v = unit stress in transverse steel in pounds per square inch; T and V = total thrust and shear; $a\,b$ = effective area of web in resisting tensile forces (a being taken along axis of hinge); $j\,d$ = lever arm of internal resisting couple; and A_v is the transverse steel area contained in $a\,b$. With $a = 8\,D''$ in which D'' = diameter of longitudinal hinge bars (2); $j = 0.9$, and p_v, the transverse steel ratio, $= \frac{A_v}{a\,b}$, the formula for p_v becomes,

$$p_v = \frac{T\left[\frac{\tan\theta}{2} + (8.9)\left(\frac{D''}{d}\right)\left(\frac{V}{T}\right)\right]}{8\,D''\,b\,f_v} - 0.005 \quad \text{............(5)}$$

For covered bars, the term $\left[(8.9)\left(\frac{D''}{d}\right)\left(\frac{V}{T}\right)\right]$ does not need to be included provided the bars are adequately anchored; and

$$p_v = \frac{T\,(\tan\theta)}{16\,D''\,b\,f_v} - 0.005 \quad \text{.....................(6)}$$

from which p_v may be determined for hinges with covered bars.

Considère Hinges.—Considère hinges may be designed by obtaining the percentage of longitudinal and spiral steel from current spiral column formulas and then by limiting the deformation in the extreme fibers at maximum rotation to a permissible proportion of the ultimate value. The following design formulas for symmetrically reinforced hinges have been developed previously (11) by the writer on the assumption of a linear distribution of strain, a parabolic (2d degree) distribution of concrete stress, and a limit of 50% of the ultimate compressive unit deformation in the concrete at the rotation ϕ:

$$\frac{P}{A_g} = \frac{5}{12} f_c'\,k\left(\frac{d}{d'}\right) + \frac{E_s\,p_g\,\epsilon_c'}{4\,k}\left(2\,k - \frac{d'}{d}\right) \quad \text{..........(7)}$$

$$\frac{\phi}{h} = \frac{\epsilon_c'}{2\,k\,d} \quad \text{........................(8)}$$

and

$$f_s = \left(\frac{1-k}{k}\right)\frac{\epsilon_c' E_s}{2} \quad \text{.......................(9)}$$

in which A_g = the gross area of a spiral column or a Considère hinge; d = the effective depth, and d' the full depth, of the hinge section; E_s = Young's modulus for steel; ϵ_c' = ultimate compressive unit deformation of concrete; and k = ratio of distance from the neutral axis to the outer fiber. Occasionally, the tensile stress, occurring in the steel during rotation, is of prime importance and must be checked in order to obtain the proper grade of longitudinal steel. A design chart for the use of Eqs. 7, 8, and 9 has already been developed (11) for a deformation limit of 50% of an assumed ultimate of 0.004 in. per in. in the concrete.

The quantity of spiral steel should be large enough to act as diagonal tension reinforcement in a member under combined shear and direct stress. With the section primarily under compression, the maximum shearing unit stress may be taken at 1.5 times the average, without appreciable error. Thus, the formula for use of the spiral as web reinforcement may be taken as,

$$\frac{2\,s\,b}{(D')^2} = \frac{N\,\pi\,f_v}{\left(\frac{T}{2\,A_g}\right)\left[\sqrt{1+9\left(\frac{V}{T}\right)^2}-1\right]-C'\,f_c'} \quad \text{.........(10)}$$

in which s = pitch of spiral; b = breadth of hinge; D' = diameter of rod forming the spiral; A_g = gross area of hinge; f_v = stress in spiral; $C'\,f_c'$ = allowance for concrete in diagonal tension; T and V = total thrust and shear, respectively; and N = number of spiral units in A_g. With the steel stress limited to 16,000 lb per sq in. this becomes

$$\frac{s\,b}{(D')^2} = \frac{50{,}300\,N}{\left(\frac{T}{A_g}\right)\left[\sqrt{1+9\left(\frac{V}{T}\right)^2}-1\right]-2\,C'\,f_c'} \quad \text{.........(11)}$$

from which the spacing and diameter of the rod may be checked against the values obtained from the design as a spiral column under direct thrust.

Considère Rollers.—Mr. Scott (12) recommends that the mid-section of the Considère roller (Fig. 6) be designed with $\frac{2}{3}\,f_c'$ as the limit for the maximum compressive stress on the cross-sectional area. The appropriate radius will be determined from the usual relationship between load and radius for rollers acting on a plane ($P = C\,r$, in which P = load in pounds per linear inch, r = radius of roller, and C = a constant given by Mr. Scott as 342 for a concrete mixture at least as rich as 1 : 1.5 : 3). The line $P = 350\,r$ is plotted in Fig. 9, which provides a factor of safety of about 4 on the ultimate of the tests shown and greater than 2 as compared to the loads at initial cracking.

Messrs. Taylor, Thompson, and Smulski suggest that the percentage of spiral steel (or hooping) should be sufficient to resist a total tension equal to one third of the total concentrated reaction acting on the roller, with the use of lead sheets, of appropriate width, between the contact faces as a substitute for the curvature.

It has been shown (14) that two equal and opposite concentrated forces acting along a diameter of a roller develop a uniform tensile stress of $\frac{P}{\pi r}$ pounds per square inch on a plane containing the contact lengths. Therefore, the total tensile force normal to the diametral cross section containing the contact lengths would be $\frac{2P}{\pi}$. This is the criterion for initial tensile cracking beyond which a truly circular roller can no longer be considered as acting as a unit within the elastic range of stress.

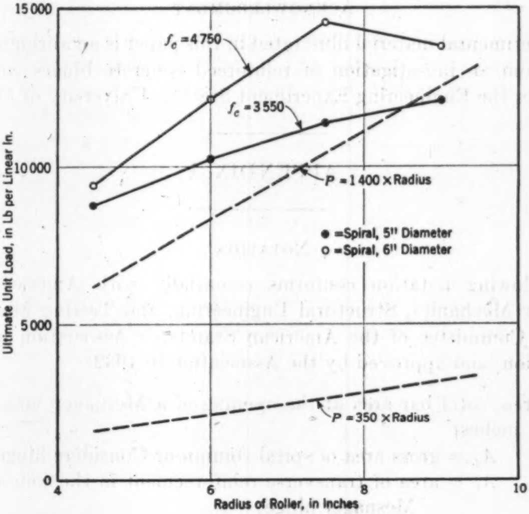

FIG. 9.—ROLLER RADIUS VERSUS UNIT LOAD ON THE CONTACT LENGTH OF CONSIDÈRE ROLLERS

Practical design for such rollers requires that the roller be narrow as shown in Fig. 6. This changes the uniform tension on the vertical section to a distribution having a unit stress higher than $\frac{P}{\pi r}$ near the top and bottom. When the breadth is $\frac{4r}{\pi}$, the distribution of tensile stress on the vertical section will be the same as that of the compressive stress on the central horizontal cross section and also not seriously different from a uniform distribution. A further reduction in the breadth would be questionable since the concrete (prior to cracking) or the steel (at the ultimate load) soon would become excessively over-stressed near the top and bottom. This may be noted in Fig. 9, since the breadth varies from 1.56 r for the 4.5-in. radius to 0.78 r for the 9-in. radius. Aid in resisting this action, after tensile cracking, is supplied by the grid stirrups placed at top and bottom of the spiral.

In general, it appears best to supply sufficient spiral steel to resist two thirds of the concentrated reaction coming to one contact length, with the limitation of $\frac{4r}{3}$ for the minimum breadth. The addition of grid stirrups at top and bottom of the spiral, each to resist a tensile force of one third of the concentrated reaction, would be a desirable safeguard against the possible non-uniformity of tensile stress distribution. It is apparent from Fig. 7(*a*) that it would be well to limit the maximum inclination of the resultant to not more than 0.05 radian with the center line of the spiral, and from this inclination determine the diameter (or height) of the roller with the aid of the known horizontal movement.

ACKNOWLEDGMENT

The experimental material illustrated in this paper is an abridgment of data resulting from an investigation of reinforced concrete hinges conducted by the writer for the Engineering Experiment Station, University of Maryland.

APPENDIX I

NOTATION

The following notation conforms essentially with American Standard Symbols for Mechanics, Structural Engineering, and Testing Materials compiled by a Committee of the American Standards Association with Society representation, and approved by the Association in 1932:

A = area; total bar area at the opening of a Mesnager hinge, in square inches:
  A_g = gross area of spiral column or Considère hinge;
  A_v = area of transverse reinforcement in the web area $a\,b$ of a Mesnager hinge;
a = effective length for tensile resistance along the common axis of members adjoining the hinge;
b = breadth of the hinge section;
C = coefficient; experimental coefficient applied to the radius of Considère rollers:
  C' = coefficient of the ultimate strength f_c' for diagonal tension resistance of concrete;
D = diameter of spiral, in inches:
  D' = diameter of the rod forming the spiral;
  D'' = diameter of the longitudinal bars;
d = depths; distance from the outer fiber to the point of application of the resultant tensile stress of a reinforced concrete member:
  d' = full depth of hinge section;
E = elasticity modulus, in pounds per square inch:
  E_s = modulus of elasticity for steel;

f = unit fiber stress, in pounds per square inch:
    f_c' = ultimate strength of plain concrete;
    f_s = maximum compressive steel stress in a Mesnager hinge, or the tensile stress in a Considère hinge;
    f_s' = direct compressive stress in Mesnager hinge bars computed on an assumption of pin-connected ends;
    f_v = stress in the transverse reinforcement of Mesnager hinges or in the spiral of a Considère hinge;
h = height of hinge opening, in inches;
j = ratio of the lever arm of the resisting couple in a reinforced concrete beam to the distance between the outer compressive fiber and the point of application of the resultant tensile stress;
K = a substitution factor in Eq. 1 (see Eq. 2*a*);
k = ratio of the distance from the neutral axis to the outer fiber of a reinforced concrete beam to the distance from the outer fiber to the point of application of the resultant tensile stress;
L = a substitution factor in Eq. 1 (see Eq. 2*c*);
l = length of a bar not completely embedded in concrete at the hinge opening, in inches $\left(\frac{l}{r} = \text{slenderness ratio}\right)$;
N = number of spiral units within an area, A_g;
P = total load on a hinge or roller, in pounds;
p = steel ratio:
    p_v = transverse steel ratio in Mesnager hinges;
    p_g = longitudinal steel ratio in spiral columns, based upon gross area;
Q = a substitution factor in Eq. 1 (see Eq. 2*b*);
R = a substitution factor in Eq. 1 (see Eq. 2*d*);
r = radius of gyration of a bar at the hinge opening, in inches $\left(\text{slenderness ratio} = \frac{l}{r}\right)$;
s = pitch of a spiral, in inches;
T = total thrust component of the load P, in pounds;
V = total shear component of the load P, in pounds;
ϵ = unit deformation, in inches per inch:
    ϵ_c' = ultimate compressive unit deformation of concrete;
θ = angle between a sloping bar and the center line of a Mesnager hinge, in degrees;
ϕ = the rotation of a hinge, in radians.

APPENDIX II

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DISCUSSION

A. A. Eremin,[3] **Assoc. Am. Soc. C. E.** (by letter).—Interesting information concerning Mesnager and Considère hinges is contained in this paper. However, some problems of hinge design still remain to be clarified. It would be useful to know the assumptions on which the equations for computation of stresses in the hinges cited by the author were based.

Evidently Eq. 1 can be applied only in the computation of the stresses within the elastic limit. Nevertheless, the author used Eq. 1 for computing the ultimate stresses in his diagram in Fig. 2. Furthermore, Eq. 1 was developed with the assumption that steel bars in the hinge form a truss and that the concrete part of the hinge was considered as a rigid member. Actually, the concrete, even if it is reinforced with steel hoops or transverse wiring, is elastic. This is a serious limitation of Eq. 1, and it suggests some need for simplifying the equation by omitting the minor factors.

Mr. Mesnager has limited the bar slenderness to a maximum ratio of $l/r = 40$ and has considered the concrete cover as a means of protecting the steel bars from corrosion. If slender steel bars in the hinge were used and their strengths varied with the stiffening effect of the concrete cover as shown in Figs. 3 and 4, then that hinge would lose the characteristic features of the Mesnager hinge and its design would be based on an entirely different principle.

The advantage of the compressive concrete members reinforced with spiral reinforcement is in the increased strength of the concrete core formed by the spiral reinforcement. Eqs. 7 to 11 however were developed for a rectangular section considering concrete outside spiral reinforcement as a part of the section. At high stresses the concrete cover in the Considère hinge is generally cracked and peeled off. The highest efficiency of the Considère hinges was obtained when concrete cover was omitted. In the temporary Considère hinges used for construction purposes the concrete cover outside the spiral reinforcement is not required. In the finished structures the temporary Considère hinges are generally cast in the permanent concrete members. The Considère temporary hinges, without concrete cover, are especially convenient in the construction of reinforced concrete arch bridges.

George C. Ernst,[4] **Assoc. M. Am. Soc. C. E.** (by letter).—The comments of Mr. Eremin are appreciated and the writer will discuss the various points in the order in which they were presented.

No attempt was made to provide the complete development of the equations nor to state the usual structural assumptions, inasmuch as such material is obtainable elsewhere from readily available sources (2) (3) (11). Mr. Eremin may refer to such sources for the more comprehensive treatment of the analyses.

A careful study of the work of Messrs. Parsons and Stang (2), as well as that of the writer, will reveal that no claim has been made as to the exactness with which Eq. 1 represents the actual stresses within the hinge. The writer sees no objection to the development and use of a simpler formula, but it should be recognized that Eq. 1, by providing the maximum stresses, may be utilized to guard against the effect of repeated rotation when considered critical, as suggested by Admiral Moreell (3). In his comprehensive investigation of all types of hinges (22), Mr. Kluge took advantage of short hinge openings, with consequently small values of $\frac{L}{r}$ for the Mesnager hinge in order to assume that the full yield point of the steel might be developed. Tests by Mr. Kluge appear to show that repeated rotations are not critical in ordinary construction. The major objection to Eq. 1, as noted by the writer, is that it shows a nonexistent advantage in the reduction of the maximum stress for the larger values of $\frac{L}{r}$, due to the increased flexibility of the bars. If a proper limit is placed on $\frac{L}{r}$ for various grades of steel, Eq. 1 will yield conservative values.

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The writer wishes to emphasize that, although Eq. 1 was developed for stresses within the elastic limit, nevertheless it may be compared satisfactorily with experimental data at the ultimate since the load at first yielding is not critically different from that at the ultimate for structural members subjected to compression and shear, provided that a proper limit is placed upon the slenderness ratio.

The comments on Fig. 3 with regard to the change in action of the Mesnager hinge due to the concrete covering are quite true. With a limit on $\frac{L}{r}$ consistent with the grade of steel used, an adjustment for the effect of the covering may be made as suggested by the writer without the need for expressing exactly the stress conditions. The added resistance to rotation caused by the covering was found to be negligible in the writer's work as well as in that of Messrs. Parsons, Stang, and Kluge. Fig. 4, however, does not pertain to the Mesnager hinge although Mr. Eremin's discussion implies that it does. The writer believes that the formulas presented for the Considère hinges of Fig. 4 approximate actual conditions more closely than does Eq. 1 for the Mesnager hinge.

In so far as the Considère hinge is concerned, it should be recognized that such hinges should never be used at stresses that would crack and peel off the cover over the spirals, although Mr. Eremin states that such a condition is generally true. Actual designs (9) (10) as well as tests (11) reveal that the working stresses and deformations on such sections are well below those at which cracking and peeling of the cover exist even at the outer fibers of the high compression side when subjected to normal rotations. It seems a bit fantastic to talk of the increased strength of the core of a spiral and then choose a working stress which prevents the utilization of such properties. As a matter of fact, the plastic yielding of the Considère hinge, if subjected to such extreme conditions over the entire section, would prohibit the use of the units (11). The recommendation of the writer to keep within an allowable limiting deformation at the outer fiber of the high compression side when subjected to maximum rotation is conservative and permits the greatest use of the deformation ability of the concrete that should be permitted. The proposed value of 0.002 in. per in. is a limit in complete accord with field tests (10). Such a limit for the outer fibers, however, will fall far short of cracking off the covering over the spiral.

Unless the hinge contains only one spiral, it will be simpler to cast the hinge in a rectangular section. This would normally be the case, inasmuch as three or more spirals are generally required in each hinge for an arch of the size requiring such construction. In any event, the spiral remains virtually inactive, except as shear reinforcement, for the working stresses and deformations used in the past, or for the limiting deformation recommended by the writer.

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ANALYSIS OF LEGAL CONCEPTS OF SUBFLOW AND PERCOLATING WATERS

BY C. F. TOLMAN[1] AND AMY C. STIPP[2]

WITH DISCUSSION BY MESSRS. DONALD M. BAKER, SAMUEL C. WIEL, HYDE FORBES, RONALD B. HARRIS, EDWARD F. TREADWELL, O. E. MEINZER, M. R. LEWIS, BAYARD F. SNOW, HAROLD CONKLING, AND C. F. TOLMAN AND MRS. AMY C. STIPP.

SYNOPSIS

In this paper are presented some of the problems confronting the courts in attempting to determine types of ground-water occurrence according to the current legal classification of subsurface waters, which calls for differentiation between ground water flowing in definite subsurface water courses and so-called "percolating waters."

The complexity of a problem involving property rights to unseen, slowly percolating ground water is appreciated. The writers are not qualified to discuss the legal questions involved. However, the attempt is made to analyze, from the viewpoint of the ground-water hydrologist, some of the erroneous hydrologic concepts involved in the present legal classification of underground water and in past rulings following this classification. Elementary principles of hydrology governing the occurrence and movements of free ground water are presented to explain the fallacies in the legal concepts of "subsurface stream flow" and "percolating waters." For simplicity, principles of artesian flow, although of primary importance where confined water transmits pressure, are not discussed in this brief paper.

The writers appreciate that much of the material contained herein has received comprehensive treatment in a paper by Harold Conkling,[3] M. Am. Soc. C. E. However, it is believed that the subject warrants further independent treatment.

NOTE.—Published in December, 1939, *Proceedings*.

[1] Prof. of Economic Geology, Emeritus, Stanford Univ., Stanford University, Calif.

[2] Tucson, Ariz.

[3] "Administrative Control of Underground Water: Physical and Legal Aspects," by Harold Conkling, *Transactions*, Am. Soc. C. E., Vol. 102 (1937), p. 753.

It should be noted that more than a quarter of a century has elapsed since publication of the legal textbooks cited in this paper.[4] In recent years much progress has been made in the development of the science of ground-water hydrology under the able leadership of O. E. Meinzer and his staff of hydrologists in the U. S. Geological Survey.[5]

It is believed that a classification of subsurface waters for legal purposes based on well-established hydrologic principles would do much to correct the present confused status of ground-water law and the unsound concepts regarding ground-water occurrence and motions which have been handed down, in part at least, from a period prior to the development of the science of ground-water hydrology.

INTRODUCTION

Agriculture, mining, manufacturing, and even human existence depend upon the efficient utilization of ground-water supplies where severe deficiency in surface water exists. Development of the vast ground-water resources of the semiarid regions of the United States is subject to the establishment of legal rights to water supply; hence, the application of ground-water law affects the material welfare of all who are dependent upon industries using large quantities of water where surface-water supply is insufficient.

For legal purposes subsurface water (without considering artesian water) is divided into two main classes, based on a difference (erroneously assumed) in method of transmission through the ground, namely: (1) "Percolating waters"; and (2) "subsurface water courses." The term "percolating waters" has been defined in numerous ways and applied to embrace ground-water occurrence under varying hydrologic conditions. By some courts the term is restricted to "diffused waters" in privately owned lands; and by others it has been extended to include all subsurface water not proved to occur in "underground water courses" with definite subsurface "bed and banks." Likewise, the term "subsurface water course" has no fixed meaning and many conflicting interpretations have been offered by courts of law. It is not surprising that this lack of a sound fundamental legal classification of subsurface waters has resulted in confused ideas regarding the occurrence and motions of water underground.

In many jurisdictions rights to ground-water supply still depend upon the court's differentiation of the two legal "classes" of subsurface water. Some jurisdictions grant riparian or appropriative rights only for ground water in "subsurface water courses," and the so-called "percolating waters" belong exclusively to the owner of the land. Other states have recognized that an attempt to make such distinction is impracticable from both the economic and hydrologic viewpoints.

This question of differentiation, then, is of paramount importance in legal proceedings. For this reason, and because many curious and erroneous notions have arisen in the attempt to separate ground water in subterranean "water courses" from "percolating waters," discussion in this paper is limited to two types of ground-water occurrence: (1) Ground water moving as a concentrated current bounded by relatively impervious subsurface "bed and banks"; and (2) the general body of ground water adjacent to the subsurface stream.

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[4] "Treatise on the Law of Irrigation," 2d Ed., by C. S. Kinney, Bender-Moss Co., San Francisco, 1912; also "Water Rights in the Western States," 3d Ed., by S. C. Wiel, Bancroft-Whitney Co., 1911.

[5] See especially *U. S. Geological Survey Water-Supply Papers: No. 494*, "Outline of Ground-Water Hydrology," by O. E. Meinzer, 1923; *No. 489*, "The Occurrence of Ground Water in the United States," with a discussion of principles, by O. E. Meinzer, 1923; and *No. 638-C*, "Methods of Estimating Ground-Water Supplies," by O. E. Meinzer.

Other typical occurrences of ground water are mentioned briefly in order that the reader may appreciate that this discussion is strictly limited in scope. Part I, a brief review of principles underlying the occurrence of ground water in "subsurface water courses" and the interrelation of surface flow, subflow, and "percolating waters" under various hydrologic conditions, is presented in order that the analysis of legal rulings in Part II may be understood by readers not familiar with these fundamental natural laws. The necessity of a knowledge of the elementary principles of ground-water hydrology (see Appendix, Item 1) as a basis for legal rulings regulating the use of subsurface water is emphasized, and it is hoped that the simplicity of the general principles outlined herein may encourage lawyers to review the present-day science of ground water. Those versed in ground-water hydrology may be interested chiefly in Part II and the comparison between man-made laws regulating subsurface-water supplies and the laws of nature governing occurrence and movements of water underground.

PART I.—OCCURRENCE AND MOVEMENTS OF WATER UNDERGROUND

The term subsurface water includes all types of water occurring below ground surface. The water of the zone of saturation is ground water. Suspended water (see Appendix, Item 2) occurs in the undersaturated zone above the water table (ground-water level) and some of this water (gravity water) is in transit to the ground-water body. Although the efficient osmotic processes of plant roots can extract moisture from the soil, man can divert and utilize only ground water.

The zone of suspended water (zone of aeration) always exists above a body of ground water filling alluvial material unless saturation extends to ground surface. Surface water seeps slowly to the water table at a rate depending largely upon the permeability of the formations through which it passes. Gravity water (influent seepage) descends vertically over films of water attached to granular materials in the zone of suspended water.

Percolation.—Percolation is curvilinear flow around the grains and through the interconnected pores of saturated pervious materials. In granular materials (chiefly alluvial deposits) interstices are interconnected in all directions and free ground water (see Appendix, Item 3) moves in the direction of the slope of the water table. The rate of percolation of free ground water varies directly with water-table slope or hydraulic gradient and permeability of the water-conducting material.

The scientific meaning of the term percolation should not be confused with the legal application of the term "percolating waters" as distinguished from flow in subsurface water courses. "Subsurface streams," "percolating waters," and "artesian waters," as defined by lawyers, all move by percolation in interconnected openings of capillary size.

Influent and Effluent Conditions.—Ground water is supplied by influent seepage from rainfall, surface flow, impounded surface water, and from irrigation water and is discharged by effluent seepage, springs, and by plant roots if they reach the capillary fringe above the water table (see Appendix, Item 4). The reactions between surface and subsurface "stream flow" and "percolating waters" depend upon whether influent or effluent water-table conditions exist. To illustrate the phenomena accompanying influent and effluent seepage one may start with the equilibrium that exists where influent conditions merge into effluent conditions and vice versa. The surface stream neither receives water from, nor loses water to, the subsurface water body. In such a case the stream level and the water table are shown as a horizontal line in a cross section at right angles to surface flow (Fig. 1), and there is no movement of ground water toward or away from the stream.

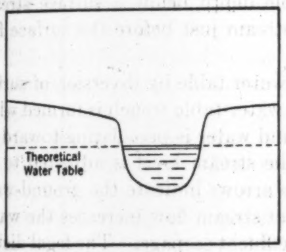

FIG. 1.—Condition of Static Equilibrium Between Water Table and Surface Stream

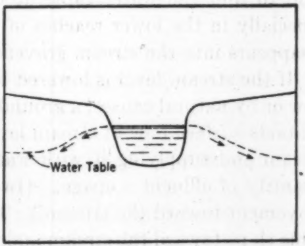

FIG. 2.—Ground-Water Mound in Contact with Stream Flow

If the ground-water level in the alluvial material is lowered by pumping or by natural causes, the surface stream loses water and becomes influent, the water table slopes away from the stream, and a ground-water mound is developed in contact with surface flow. In Fig. 2 the stream and the water table are influent. The direction of the ground-water movement is shown by arrows. If the ground-water level is lowered below stream bed, the ground-water mound is not in contact with surface flow. The material between the mound and the stream bed is not completely saturated and a column of influent seepage transmits the leakage from stream bed to the ground-water mound. In Fig. 3 a

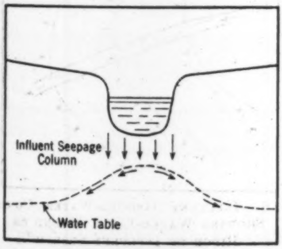

FIG. 3.—Ground-Water Mound Not in Contact with Surface Stream

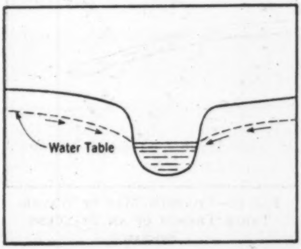

FIG. 4.—Ground-Water Trench Which Supplies Water to Surface Stream and Indicates Effluent Condition

column of influent seepage is developed below surface flow. After the mound has lost contact with the surface stream, further lowering of the water table does not increase the rate of influent seepage.

Apparently lawyers do not generally appreciate the fact that stream flow occurs over nonsaturated gravels through which water seeps from the surface to the water table. The assertion is made that "the water from the surface stream must necessarily fill the loose, porous material of its bed to the point of complete saturation before there can be any surface flow." Often the stream bed is rendered relatively impervious by silt deposited with receding flood flows or by chemical cementation, and subflow occurs only at some distance below the surface stream, supplied by slow influent seepage. In general, materials below stream bed are not uniformly pervious and such conditions do not favor development of water-table mounds in contact with surface flow. It is not uncommon to find the water table at considerable depth below a surface stream, especially in the lower reaches of a desert stream just before the surface flow disappears into the stream gravels.

If the stream level is lowered below the water table by diversion of surface flow or by natural causes, a ground-water or water-table trench is formed which contacts surface flow at stream level. Ground water is percolating toward the stream and supplying it with water, and the stream level is adjusted to the quantity of effluent seepage. (In Fig. 4 the arrows indicate the ground-water movement toward the stream.) Diversion of stream flow increases the water-table slope toward the stream and increases effluent seepage. The legal dictum that a surface stream is "supported" by subflow is applicable only under effluent conditions.

Influent and Effluent Conditions as Shown by Ground-Water Contours.—The most satisfactory method of determining the direction of ground-water flow is by the construction of a water-table contour map. Direction of percolation is at right angles to the ground-water contours. Influent and effluent conditions are given graphic representation on such maps. Effluent conditions along a stream course are shown by an upstream bend of the contours (as in Fig. 5), indicating the size and shape of the ground-water trench.

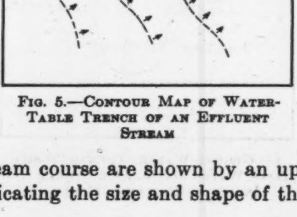

FIG. 5.—CONTOUR MAP OF WATER-TABLE TRENCH OF AN EFFLUENT STREAM

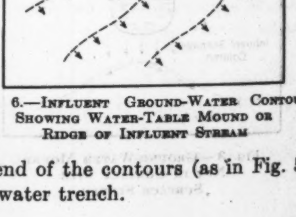

FIG. 6.—INFLUENT GROUND-WATER CONTOURS SHOWING WATER-TABLE MOUND OR RIDGE OF INFLUENT STREAM

Influent conditions and the development of a ground-water mound or ridge are indicated in Fig. 6 by a downstream bend of water-table contours.

Effects on Surface Flow of Pumping from Influent and Effluent Subflow.—Under influent conditions draft on subflow by pumping does not increase stream seepage loss unless the slope of the water table in contact with the surface stream is increased. This occurs when the "cone of pumping depression" (see Appendix, Item 5) contacts surface flow. If the water table is lowered so that it loses contact with the surface stream, increased draft accompanied by further lowering of the water table does not increase the rate of influent stream seepage.

Under effluent conditions, pumping draws into the well ground water percolating toward the surface stream. The effect on stream flow depends on the ratio of effluent seepage thus diverted to the total effluent seepage feeding the stream. Diversion of surface flow will increase effluent seepage into the stream, at least temporarily, and will lower the water table adjacent to the stream by steepening the ground-water trench.

The attention directed by the courts to the effects on surface flow produced by pumping from a subsurface stream makes it advisable to discuss these pumping effects further. Pumping from a "stream" of ground water or from any free ground-water body produces a cone-shaped depression of the water table around the well. This cone of pumping depression expands until it intercepts a quantity of ground-water flow equal to the amount pumped. Thereafter the cone of depression remains approximately constant, but the effect of interception of ground-water flow (area of diversion) spreads out in the downstream direction of water-table slope. It has been stated in legal proceedings that the ground-water level responds to pumping in the same way as the water level of a surface reservoir, which, of course, is not in agreement with hydraulic laws.

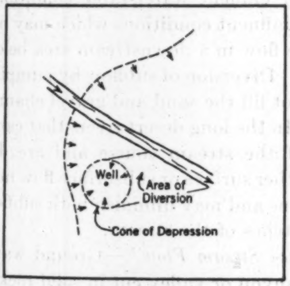

FIG. 7.—EFFLUENT WATER TABLE AND CONE OF PUMPING DEPRESSION SURROUNDING WALL

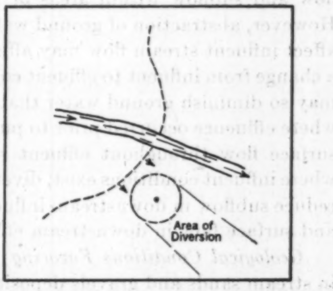

FIG. 8.—UNDER INFLUENT CONDITIONS THE CONE OF PUMPING DEPRESSION INTERCEPTS GROUND WATER MOVING AWAY FROM THE SURFACE STREAM AND AREA OF DIVERSION EXTENDS IN DIRECTION OF GROUND-WATER FLOW

Fig. 7 represents effluent conditions under which the effects of diversion of ground water will reach the surface stream even if the cone of depression does not. If the cone of depression expands as a result of overpumping the natural supply and contacts the surface stream, water is drawn directly from the stream in addition to the subflow intercepted.

Under influent conditions (Fig. 8) if the cone of pumping depression does not reach the surface stream, no pumping effect will be registered by surface flow as far downstream as influent conditions extend.

If the cone of pumping depression extends under the surface stream, water is evacuated from the materials below stream bed within the cone of depression and an influent seepage column is developed below surface flow. Regardless of whether influent or effluent conditions existed prior to the development of such a cone of depression below stream bed, the pumping well is supplied by diversion of subflow plus water furnished by the induced influent seepage from the surface stream.

It is fairly common that the flood-flow channel of streams in arid regions is a mile or more in width. If subflow occurs in a corresponding subsurface channel, it is evident that, under extreme conditions, if pumping should affect the entire width of the subsurface stream, much more water would be abstracted from the ground-water "stream" than from the relatively small cross section of surface flow, even if all surface flow is diverted by such pumping. In general, as the width and depth of the subsurface stream are usually much greater than the surface stream, the quantity of subflow diverted by the pump is larger than the quantity of surface flow abstracted.

The legal "test" for subflow, discussed in Part II, and the prevailing rule that "there cannot be any abstraction of the water of the underflow without abstracting a corresponding amount from the surface stream" are inconsistent with these principles.

Downstream Effects of Diversion Above a Change from Influent to Effluent Conditions.—Thus far this paper has been limited to reactions between surface flow and subflow within areas of influent or effluent water-table conditions. However, abstraction of ground water under influent conditions which may not affect influent stream flow may affect surface flow in a downstream area below a change from influent to effluent conditions. Diversion of subflow by pumping may so diminish ground water that it does not fill the sand and gravel channel where effluence occurred prior to pumping. In the long desert rivers that carry surface flow throughout effluent sections of the stream course and are dry where influent conditions exist, diversion of either surface or subsurface flow may reduce subflow in downstream influent portions and may diminish both subflow and surface flow in downstream effluent stretches of the stream.

Geological Conditions Favoring "Subsurface Stream Flow."—Ground water in stream sands and gravels deposited in a canyon or valley cut in solid rock is the most definite example of a "subsurface stream." Such conditions commonly exist along the mountainous headwaters of the stream. The "bed and banks" of the subsurface water course are carved in bedrock. Surface flow depends upon the subsurface stream where the latter is effluent, and subflow depends on surface flow where the surface stream is losing water by influent seepage.

After leaving the mountain uplands, most streams of the arid and semiarid regions of western United States transect a body of valley fill. It is fairly common for the larger streams to cut channels a mile or more in width through the older alluvial fill. The outermost banks cut by the stream flow are the upper flood-plain terrace faces, and the subflow "banks" might be considered to be the subsurface continuation of these terrace faces. The stream-channel and flood-plain deposits, which the stream has laid down during late depositional stages, are usually more pervious than the older valley fill and, if so, the hydrologic behavior of surface and subsurface flow shows similarities to that in rock-bound channels.

Subflow Indicated by Contour Pattern.—The ground-water contour pattern represents water-table conditions, and spacing of contours indicates the gradient of percolating water. Characteristic patterns are developed by a subsurface stream where the bed and banks are impervious bedrock and the channel deposits are pervious. Similar but less definite patterns are shown by a subsurface stream with bed and banks cut in material less pervious than the stream-channel deposits but not impervious.

The stream-laid deposits in a rock-bound channel are charged with water during flood flow and a pronounced water-table ridge is developed. The water thus stored above the low-water stages is discharged into the stream with subsiding flood flow and develops a water-table trench. Also, a rock-bound subsurface water course is usually effluent in a contracting part of the bedrock channel and develops a water-table trench; it is influent where the subsurface channel expands and a water-table mound is formed. Where bedrock is impervious the ground-water contours terminate at the bedrock banks.

Summarizing, ground-water contour patterns show the following relations to impervious bedrock boundaries of a subsurface stream:

(1) The contours terminate at the bedrock banks;

(2) Within subsurface banks the contour pattern fluctuates from influent form during rising water stages to effluent shape during subsiding water stages; and

(3) A decrease in cross-sectional area of subsurface channel develops an effluent contour pattern, and an increase in cross-sectional area produces the influent pattern.

Comparing these contour patterns with patterns developed by a subsurface stream confined between banks of material less pervious than the stream-channel deposits but not impervious, one finds that the ground-water contours bend rather than terminate at the banks of the subsurface channel. When flood flow produces a pronounced mound in the subsurface stream, the water-table contours beyond the subsurface banks are also bent into a less pronounced influent shape, indicating slight influent movement from subflow into the older valley fill. Likewise, where a sharp ground-water trench is produced by drainage of saturated stream gravels during low-water stages of surface flow, the contours beyond the subsurface banks are drawn into the effluent form and slight effluent movement is developed from the older alluvium into the subsurface stream channel (see Figs. 9 and 10).

The boundaries of such a subsurface stream course are indicated by:

(1) A bend in ground-water contours at subsurface banks;

(2) Change of contour pattern with change from high-water to low-water stage of surface flow; and

(3) Changes in contour pattern with fluctuation in cross-sectional area of subsurface channel.

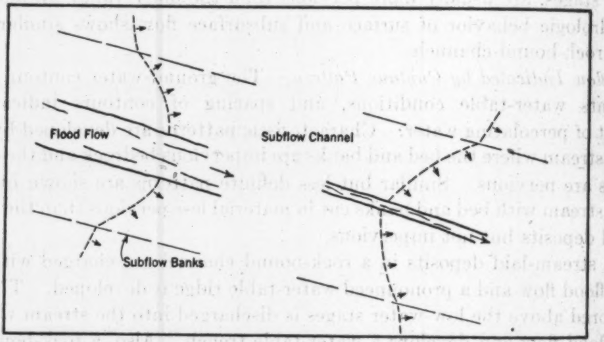

Fig. 9.—Pronounced Ground-Water Mound in Subflow Channel Produced by Flood Flow, with Contours Beyond Less Pervious Banks Modified into Influent Shape by Ground-Water Movement into Older Alluvium

Fig. 10.—Drainage of Stream Gravels with Low-Water Surface Flow, Producing Pronounced Trench in Subflow Channel with Contours Beyond Subsurface Banks Drawn into Effluent Shape Due to Slow Effluent Movement from Older Alluvium

The interpretation of contour pattern as indicating the extent of subsurface flow has been emphasized because the water-table contour pattern is the simplest and most easily recognized criterion that can be thus applied. Unfortunately, water-table maps are seldom constructed to the scale and in the detail required for such interpretation. If physical conditions render the construction of special detailed maps difficult and costly, other methods of investigation may be applied. Some of these are mentioned without further comment:

1. Topographic: Recognition of subsurface stream course by topographic features, especially stream terraces.

2. Geologic: (*a*) Differentiation of older alluvium and recent stream deposits by means of samples taken at the surface and from wells; and (*b*) location of "bed and banks" cut in older alluvial deposits by subsurface projection of flood-plain terrace faces or by shafts and drifts to expose the subsurface banks.

3. Hydrologic: (*a*) Permeability tests of samples mentioned under Item 2(*a*); and (*b*) analysis of pumping tests on wells tapping subflow and wells drawing on "percolating waters" beyond subsurface banks.

Ground Water Not Occurring in Subsurface Water Courses.—West and south of the Rocky Mountains the main streams heading in the mountains traverse a series of structural valleys formed by the depression of bedrock between uplifted mountain ranges, not by stream erosion. The principal source of water supply is mountain runoff collected by the upper reaches of the master stream and moving as surface and subsurface flow in the sands and gravels of the stream channel. Such subflow may be the principal ground-water supply of the region, and the best agricultural lands are situated on the flood-plain and terrace deposits. However, the notion that ground water must be concentrated in a restricted subsurface channel to constitute a supply sufficient for industrial or agricultural use, entertained by many well drillers and apparently accepted by some lawyers, is in error. There are many occurrences of ground water of great economic importance that cannot be classified as subsurface water courses. In permeable granular deposits ground water percolates through interconnected pores between grains throughout the body of the pervious material. In dense impermeable rocks it moves through the fracture systems. In lava containing large openings produced by flowage and fracture and in cavernous limestone ground water flows freely through large tubular openings. Each of such typical occurrences is distinctive and the type of openings carrying ground water should be determined before an attempt is made to interpret the interrelation between ground water and surface water.

In some areas ground-water supplies in lava and limestone are important; in others the only available ground water may occur in fractures. In some sections of the United States extensive confined (artesian) aquifers yield copious supplies. The alluvial deposits of the semiarid western United States contain large supplies of ground water that cannot be classed as subsurface stream flow, and where restricted subflow does occur it may be of minor importance.

The bedrock of large intermontane valleys or basins (see Appendix, Item 6) of southwestern United States is generally covered with a thick deposit of alluvium derived from the surrounding mountain ranges. The valley fill consists of coalescing alluvial fans and cones, and either the central lake or playa deposits of undrained basins or the sand and gravel deposits of the master stream which drains the basin. The clayey playa deposits do not yield large ground-water supplies, but the pervious channel deposits of axial streams usually carry subflow and are important water bearers.

Where the alluvial cones flanking these semiarid basins are large and the adjacent mountains furnish sufficient water, the principal subsurface supplies may be held in the alluvial cones. Where the stream draining the valley has not built up thick pervious channel deposits, there will be no large subflow associated with the surface stream in the center of the valley. The southern part of Owens Valley, California, is a good example of such conditions.

The lower or distal parts of alluvial cones may consist of fine, relatively impervious material which separates the ground water in the cones from that of the longitudinal valley. Alluvial cones also commonly contain artesian water. An understanding of the hydrologic characteristics of alluvial cones and of the principles governing the flow of artesian water[6] is essential for the solution of problems involving the effects of pumping from alluvial cones.

The foregoing condensed summary of a few elementary principles of ground-water hydrology offers a basis for understanding the geologic and hydrologic problems involved in the determination of the type of ground-water occurrence and in interpretation of reactions between surface and subsurface water bodies. Legal decisions made with inadequate comprehension of these physical laws may not be applicable to the particular conditions existing in the area in controversy; nor can they result in efficient regulation of the ground-water resources. Some of the difficulties encountered by the courts because of lack of understanding of hydrologic principles are presented in Part II.

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[6] "Ground Water," by C. F. Tolman, Chapters XI and XII, McGraw-Hill Book Co., Inc., 1938.

PART II.—ANALYSIS OF A FEW RULES OF LAW

Many states, especially those of the semiarid West, have abandoned the long-accepted doctrines of water law applicable to regions of plentiful surface water supply and have attempted to establish rules better adapted to physical conditions in areas of low rainfall and intermittent stream flow.

Because of keen competition for limited surface and subsurface water resources, numerous water-rights cases, involving many technical points of law, have come into the courts. It is not the purpose of this paper to discuss these technical legal questions but merely to analyze some of the legal concepts and rules regarding subsurface water and to compare them with the behavior of underground water as deduced from modern scientific study.

In the legal literature the idea has often been expressed that occurrence and movements of subsurface water are incomprehensible and unpredictable. In a textbook on water law published in 1912,[7] it is stated that it is difficult to prove the existence of a subsurface water course because "the physical laws governing underground water and its subterranean progress [are] changeable and uncontrollable in character and subject to secret and incomprehensible influences."

It is true, indeed, that an accurate determination of the quantity of ground water available may be impossible because of insufficient data, but in many cases the critical data can be collected by detailed investigations. In no case are the physical laws "irregular or unknowable"; nor is ground water subject to "secret and incomprehensible influences." The physical laws governing the movements of ground water are well established, and relatively simple engineering investigations will determine the direction of ground-water motion, and geological observation will determine the type of occurrence.

The practice of most of the law courts in the past has been to differentiate sharply between the occurrence of ground water in subsurface water courses and as "percolating waters," applying different rules to regulate the use of ground water in these two supposedly different types of occurrence. In most jurisdictions riparian or appropriative rights to ground water can be upheld only if its occurrence as a subsurface water course or "lake" can be established. Several states of the West, however, have extended rights of appropriation to occurrences of "percolating waters," and, in general, the trend seems to be toward application of equal rights to all bodies of subsurface water subject to development.

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[7] "Treatise on the Law of Irrigation," 2d Ed., by C. S. Kinney, Bender-Moss Co., San Francisco, 1912, p. 2116.

As explained in Part I, all ground-water bodies in granular materials, whether in defined subsurface channels, artesian systems, or in extensive subsurface "reservoirs" (see Appendix, Item 7), are percolating, except for rare instances of more rapid turbulent flow where a steep hydraulic gradient is developed in coarse, sorted gravel. Since analysis of the legal concepts regarding "percolating waters" involves the more elementary principles of ground-water hydrology, this legal class will be presented first.

"PERCOLATING WATERS"

The most curious concept expressed in legal literature is that of "diffuse percolations," which have been described as "vagrant, wandering drops moved by gravity in any and every direction along the line of least resistance." These waters are supposed not to "contribute, or are not tributary, to the flow of any definite stream or body of surface or subterranean waters."

As a matter of fact, there is no "diffused water" below the water table. Ground water completely fills all the openings in the material through which it percolates. No type of movement in natural deposits, either of ground water or of water seeping from ground surface to the water table, is comparable to that described for "diffuse percolations." The general acceptance of this concept of diffusion of percolating water by jurists has given rise to erroneous hydrologic dicta regarding the relation of "percolating waters" to other types of ground-water occurrence.

According to legal terminology, "percolating waters" have been variously defined and described as:

> "* * * those oozing or percolating through the soil in varying quantities and uncertain directions; water flowing, seeping, or circulating in minute particles beneath the surface of the ground, without banks or defined channels, and whose course is invisible and unknown * * *."[8]
>
> "* * * those waters which slowly percolate or infiltrate their way through the sand, gravel, rock or soil, which do not then form a part of any body of water or the flow of any water course, surface or subterranean, but which may eventually find their way by force of gravity to some water course or other body of water, with whose waters they mingle, and thereby lose their identity as percolating waters."[9]

It is evident that none of these descriptions is a correct representation of either the legal application of the term or the scientific meaning. Percolating waters as defined in the preceding quotations might be interpreted as gravity water seeping through the zone of aeration. It is stated that these waters "do not * * * form a part of any body of water * * *" and therefore, by strict application of legal definition, could not be considered a part of the ground-water body. However, water in the undersaturated zone above the water table is not available for pumping or diversion and therefore need not be considered in a classification for the purpose of establishing rights to water supply. Obviously, the term "percolating waters" does not apply to gravity water in the undersaturated zone, although the term percolation has been used synonymously with infiltration. In the legal sense, any occurrence of subsurface water not proved to qualify for more definite classification is considered as "percolating."

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[8] 55 A.L.R. 1388, and cases cited.

[9] "Treatise on the Law of Irrigation," 2d Ed., by C. S. Kinney, Bender-Moss Co., San Francisco, 1912, p. 2150.

The old common-law rule governing the use of "percolating waters" is:[10]

> "* * * the water which is held by the soil is a portion of the soil itself, and belongs to the owner of the land * * * . This rule is not changed by the character of the material through which the water percolates * * * . So long as the water is in the condition of infiltration or percolation, it is a part of the soil * * * ."

The impractical character of this rule has been recognized and the majority of states have adopted new rulings. In Wrathall vs. Johnson[11] the question of appropriation and priority of use of subsurface water is discussed at length. The court states:

> "* * * Once a departure from or modification of the common-law rule has been made, as has been done in most of the states, we see no place for us to stop short of the rule of prior appropriation. The later cases in arid regions are tending strongly towards the doctrine of appropriation and priority of use as the surest safeguard of rights to use of water * * * ."

Subsurface Reservoir.—"Percolating waters tributary to a subsurface reservoir" are usually distinguished from "vagrant, wandering diffuse percolations." The legal concept of "tributary percolating waters" is described by Kinney:[12]

> "* * * those waters which percolate down through the gravel, porous rock, and soil from various directions until they reach some barrier of impervious rock, clay or other material, constructed either by Nature or by the hand of man, thereby forming catchment basins, and there these percolating waters, mingling with the rock, gravel, sand and soil, form what are called underground reservoirs, with the water at times rising nearly or quite to the surface of the earth * * * . These waters tributary to underground reservoirs percolate, it is true, but only in the sense that they form a vast mass of water confined in a basin filled with detritus, always moving slowly down to lower levels in the effort, in conformity with physical law, to attain a uniform level."

There is no natural differentiation between percolating water "tributary" to a subsurface reservoir and the ground water of the reservoir. The common legal view that "percolating waters" are in the process of "infiltration" is erroneous. The true character of the movement of subsurface water should be emphasized. Whereas gravity water seeps downward through granular materials of the zone of aeration, percolation of the free ground-water body is always in the direction of water-table slope and the entire percolating water body below the water table constitutes the subsurface reservoir.

The view that the ground-water reservoir may be considered a static water body has been generally accepted in the past in legal proceedings. In Hinton vs. Little[13] the court offered an objection to this concept, ruling that all ground water is percolating.

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[10] Gould vs. Eaton, III Cal. 639, 44 Pac. 319 (1896).

[11] 86 Utah 50, 40 Pac. (2) 755 (1935).

[12] "Treatise on the Law of Irrigation," 2d Ed., by C. S. Kinney, Bender-Moss Co., San Francisco, 1912, pp. 2169–2170.

[13] 50 Idaho 371, 296 Pac. 582 (1931).

> "* * * It appears to be now fairly well settled that all underground subterranean waters are percolating waters; that is, that there is more or less movement, both perpendicular and horizontal, through the earth and rocks. Therefore, whether underground waters move in a well-defined channel, either in a generally confined direction as to the points of the compass or spread out laterally, is merely a question of difference of degree * * *.
>
> "It would seem, therefore, that it is impossible to attempt to lay down one rule with regard to subterranean waters, existing more or less as a relatively stationary body of water under the ground, and subterranean waters in which there is a decided movement * * * ."

To compare any ground-water body to a static lake, as was done in City of Los Angeles vs. Hunter,[14] is not justifiable. The effects of pumping from a surface reservoir or lake are very different from those produced by pumping from a free ground-water body, where the cone of pumping depression lowers the water level only throughout an area sufficient to divert the quantity pumped and ground water is continuously percolating through the water-bearing material to replenish the supply.

The significance of the fact that ground water never occurs as a stationary water body should be stressed. Ordinarily, the subsurface reservoir is continuously receiving additions by influent seepage from rainfall and surface water bodies and is always discharging water by natural processes. In the subsurface reservoir ground water is percolating toward the discharge area; no static ground-water bodies are known to exist.

In actions at law subsurface water is presumed to be "percolating water" unless bed and banks of an underground "water course" are definitely established. The hydrologic problems involved in an attempt to distinguish between "subsurface stream flow" and "percolating waters" are discussed in the following sections.

SUBSURFACE WATER COURSE

According to legal definition a water course "consists of a bed, banks, sides, or walls, and a current of water. It is a living stream confined to a channel, usually flowing in a particular direction * * * ."[15]

"A 'stream of water' has a defined channel. It has banks, and is very distinct from the percolations of subsurface water, which oozes in veins or filters through the earth's strata. An 'underground stream' of water differs from a surface stream only with respect to its location above or below the surface."[16]

The legal concept of the geologic and hydrologic conditions determining whether a ground-water body constitutes "percolating waters" or a subsurface water course is described in Miller vs. Bay Cities Water Company, as follows:[17]

> "* * * A water course must consist of bed, banks or sides and water, and the water must be flowing in a channel or course more or less defined * * * . Waters moving by force of gravity in a valley or basin of wide

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[14] 156 Cal. 603, 105 Pac. 755 (1909).

[15] "Treatise on the Law of Irrigation," 2d Ed., by C. S. Kinney, Bender-Moss Co., San Francisco, 1912.

[16] Stoner vs. Patten, 132 Ga. 178, 63 S.E. 897 (1909).

[17] 157 Cal. 256, 107 Pac. 115, 27 L. R. A. (N. S.) 772 (1910).

> extent * * * and moving generally through the whole or through a large portion of the basin, along through the natural voids or interstices of the earth, composed of alluvial or other deposits, lying throughout the entire basin, and made up of loam, sand, gravel and bowlders, mixed together, and interspersed with irregular and broken strata or masses of clay or cemented sand and gravel, and lying in place as originally deposited by the forces of nature, do not constitute a water course, but are a part of the land, and belong to the owner of land as fully as any other constituent part of it * * * . But if such water does collect underground and flow in certain courses or channels through coarse, permeable material therein, where the existence and general course of the flowing or moving body of water can be easily determined, then the water so moving in such channels would constitute a water course, although not visible on the surface, and although the space through which the channel extends may be largely filled with the material through which the water flows."

It should be realized that, in the light of present-day knowledge, "the existence and general course of the flowing or moving body of water" can be determined as readily for the water percolating throughout an extensive "basin" or valley as for ground water flowing in a restricted subsurface channel. Whereas percolation of ground water below a surface stream is frequently restricted by fairly well defined "bed and banks," subflow often cannot be differentiated from the general ground-water body except as a water-table mound or trench below surface flow.

As a matter of fact, considerable difficulty has been encountered by the courts in delimiting subflow and in defining a subsurface water course. In City of Los Angeles vs. Pomeroy[18] the court defined the boundaries of a water course as follows:

> "* * * While a water course must have a bed and banks or sides, yet such bed may consist of any material which keeps the waters from penetrating below a certain depth, and such banks or sides may consist of any material which has the effect of confining the waters within circumscribed limits."

Every ground-water body, of course, meets the conditions of this ruling, as the limiting boundaries of any body of ground water may be defined more or less accurately. The extent of the subsurface reservoir, if not restricted to a subflow channel, is usually considered as limited to some topographic unit, such as the pervious area of a surface drainage basin or the slopes of an alluvial cone. Although the saturated water-conducting material is never uniformly permeable throughout and may be interrupted locally by lenses or layers of impervious material, the subsurface reservoir is considered to extend throughout the pervious alluvial material of the basin.

What constitutes a "known and defined" subsurface water course, then, depends upon the viewpoint of the court, and decisions have been based on various types of evidence.

In City of Los Angeles vs. Pomeroy[18] it was ruled that:

> "* * * 'defined' means a contracted and bounded channel, though the course of the stream may be undefined by human knowledge; and the

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[18] 124 Cal. 597, 57 Pac. 585 (1899).

word 'known' refers to knowledge of the course of the stream by reasonable inference."

Numerous decisions have been based on the requirement that:

> "* * * without opening the ground by excavation, or having recourse to abstruse speculation of scientific persons, men of ordinary powers and attainments would know, or could with reasonable diligence ascertain, that the stream, when it emerges into light, comes from, and has flowed through, a defined subterranean channel."[19]

In Maricopa County Municipal Water Conservation District vs. Southwest Cotton Company[20] the rule applied was:

> "* * * there must be clear and convincing proof to the satisfaction of a reasonable man, not only that there are subterranean waters, but that such waters have definite bed, banks and current, and evidence must establish with reasonable certainty the location of such bed and banks; there must be certainty of location, as well as of existence of the stream * * *."

The rulings in this case are significant. The trial court accepted the geological evidence presented as conclusive that ground water occurred in a dependent subsurface water course, fed by the influent river. However, a higher court reversed the decision, ruling, in regard to proof of a water course, that it is insufficient in Arizona to base appropriation on evidence that

> "* * * somewhere beneath the surface * * * geological theory teaches us that there are subterranean channels * * * . The decree of the trial court, so far as it adjudges plaintiff a right of prior appropriation to water of any wells not in or immediately adjacent to the surface channel of the Agua Fria River, cannot be sustained on the ground that such water comes from either dependent or independent underground channels with known and definite banks * * *."
>
> "* * * There is not a scintilla of evidence in the record from which the ordinary man, or even the trained scientist, could point out definitely a specific place where any one of the so-called subterranean water courses begins, where it ends, or how far its banks extend."

It appears, therefore, that each court must establish a ruling based on its interpretation of existing geologic and hydrologic conditions, and if courts refuse to accept scientific evidence indicating natural conditions their decisions may not be well founded.

Subflow.—In the legal sense, "underflow, subflow, or undercurrent of a surface stream" is considered as "those waters which slowly find their way through the soil, sand, and gravel constituting the beds of streams, or the lands under and adjacent to the streams which are themselves a part of subsurface streams." By strict application of the conditions given in this definition, subflow could be considered only as the ground-water mound or ridge in contact with an influent stream or the ground water adjacent to an effluent stream.

The test applied to determine whether or not ground water constitutes subflow is:[20]

> "* * * there cannot be any abstraction of the water of the underflow without abstracting a corresponding amount from the surface stream, for

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[19] Clinchfield Coal Corp. vs. Compton, 148 Va. 437, 139 S.E. 308 (1927).

[20] 39 Ariz. 65, 367, 4 Pac. (2) 369 (1931), 7 Pac. (2) 254 (1932).

> the reason that the water from the surface stream must necessarily fill the loose, porous material of its bed to the point of complete saturation before there can be any surface flow. Therefore the river bed must continue holding sufficient water to support the surface stream * * * for otherwise in drawing on the underground flow of the stream it will necessarily draw upon the water flowing on the surface."

Analyses of the definition of subflow and the determining test disclose numerous fallacies. Without going into detail, the following discussion should indicate some of the hydrologic principles involved.

Subflow as "a Part of the Surface Stream."—The waters of subflow are specifically qualified as being "a part of the surface stream." Effluent subflow or influent subflow, with ground-water mound in contact with surface flow, comply with this qualification. Obviously, effluent subflow, which is feeding the stream, may be considered to constitute a part of surface flow. An effluent ground-water body "supports" the surface stream, in that it contributes to surface flow.

A ground-water mound built up to surface stream level also might be considered a part of the surface stream, in so far as it is in contact with surface flow. A well that taps such a mound may abstract water directly from surface flow. However, where an influent seepage column occurs between surface flow and the water table, it is evident that subflow cannot be considered a part of the surface stream. In any case it cannot be said that a ground-water mound "supports" the surface stream, either in the sense that it furnishes a physical support or that it contributes to the supply of surface flow. On the contrary, the surface stream supplies all of the water of the mound.

It has been stated that diversion of subflow or influent "percolations" creates "an artificial draft upon the surface flow which draws down a part of it and weakens and injures the natural bed * * * ."[21] Numerous cases have been based on the theory that riparian owners or appropriators of surface flow are entitled to have the "percolating and seeping waters" remain undisturbed for the purpose of "supporting and maintaining" the water in the surface water body. A California court held that "a landowner cannot sink a well so near an irrigation ditch of another that the water supply would be taken from the ditch."[22] The principle involved, as stated in the legal brief, is "the right to the support to the water flowing in the ditch afforded by the seepage water escaping therefrom into the soil reasonably adjacent to the ditch."

In City of San Bernardino vs. City of Riverside[23] the following concept of "support" furnished by subflow is expressed:

> "* * * When a stream runs over porous material saturated with water, * * * the underground waters support the stream, either by upward or lateral pressure, or feed it directly * * * ."

This concept that "support" is generally afforded the surface stream by subflow is erroneous and is true only where effluent conditions exist. Obviously influent ground water furnishes no physical support to surface flow whether the ground-water mound is in contact or at some distance below the surface stream. Ground water percolates down the slopes of the water-table mound away from an influent stream and there is no "upward or lateral pressure" exerted on the water in the stream. Whether the stream is influent or effluent, the bed is not "weakened or injured" in any way by diversion of subflow.

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[21] Vineland Irrigation District vs. Azusa Irrigation Co., 126 Cal. 486, 58 Pac. 1057, 46 L. R. A. 820 (1899).

[22] Lemm vs. Rutherford, 76 Cal. App. 455, 245 Pac. 225 (1926).

[23] 186 Cal. 7, 198 Pac. 784 (1921).

The hydrologic test that

> "Drawing on the underground flow of the stream * * * will necessarily draw upon the waters flowing on the surface (because) the surface stream must necessarily fill the loose, porous material of its bed to the point of complete saturation before there can be any surface flow * * * ."

is not generally applicable. The erroneous assumption involved in the latter part of this statement is evident from previous discussion of the development of ground-water mounds by influent seepage. Pumping effects under influent and effluent conditions have also been treated in Part I. These will be reviewed briefly, however, from the standpoint of the test for subflow.

Influent and Effluent Conditions.—Surface flow responds to the test for subflow only under certain conditions of the water table with relation to surface flow. If subflow is effluent, the test is partly valid. Pumping will diminish the quantity of ground water reaching the surface stream throughout the parts of the stream course affected by the area of pumping influence (see Fig. 7). Reduction of effluent seepage feeding surface flow will be in direct proportion to the decrease in gradient of the water-table trench sloping toward the stream. Furthermore, if the cone of pumping depression reaches surface flow, water will be abstracted directly therefrom. In most cases reduction of surface flow produced by pumping subflow is small compared to the quantity abstracted from the more extensive ground-water body.

If subflow is influent, pumping therefrom will draw directly on surface flow only if the slope of the ground-water mound in contact with the stream is increased, which occurs if the cone of pumping depression extends to the surface stream. The quantity thereby withdrawn from surface flow is directly proportional to the increase in water-table slope away from the stream. As explained in Part I, the quantity thus directly diverted from surface flow is usually small compared to the quantity withdrawn from the subsurface stream.

In the common case in arid regions where the ground-water mound is not in contact with the surface stream, pumping from the influent ground-water body can have no effect on surface flow throughout the area in which stream flow is feeding water underground. Pumping from an influent ground-water body not in contact with surface flow does not increase the rate of stream seepage loss. It is not true in any case that pumping from subflow abstracts "a corresponding amount from the surface stream," although it may draw directly on surface flow.

Desert streams often carry surface flow only where ground water is forced toward the surface by contraction of bedrock channel, producing effluent seepage. Pumping of subflow under such conditions may prevent or reduce effluence downstream from the pumps. Furthermore, pumping of effluent subflow or diversion of surface flow will diminish influent subflow downstream from such diversion. On such streams subflow and surface flow are so closely interrelated as to make any distinction between the two types of flow almost impossible.

Streams in less arid regions may carry surface water throughout their course. Where restricted to narrow bedrock channels, or where the channel is cut through relatively impervious valley fill, the flow may fluctuate from influent during flood-flow stages to effluent conditions with low-water stages. Various local hydrologic and geologic conditions, also, may cause a change in the relation of water table to surface flow. The surface streams of alluvial cones are usually influent near the apex of the cone and effluent toward the lower margin.

It is evident, therefore, that determination of the relation of the ground-water body to surface flow and evaluation of pumping effects often involve geologic and hydrologic problems. Effects of pumping or diversion in upper reaches of a stream upon surface and subsurface flow in the lower stream depend upon the relation of surface flow to subflow throughout the intervening section of the stream course. It is not possible to interpret such pumping effects without investigating existing geologic and hydrologic conditions.

CONCLUSIONS

This paper has stressed some of the problems involved in court interpretations of the type of ground-water occurrence according to the present classification of subsurface water for legal purposes. Some of the erroneous legal concepts resulting from inadequate comprehension of geologic and hydrologic factors governing occurrence and movement of water underground have been discussed. In order to establish a sound classification and to formulate rulings for efficient regulation of ground-water resources, greater consideration should be given to principles of ground-water hydrology. A few general rules of law founded on well-established hydraulic engineering principles should eliminate much of the confusion that has arisen in the attempt to follow prior legal rulings based on an arbitrary classification not in agreement with occurrence of water below ground surface.

APPENDIX

DEFINITIONS

The following definitions apply to terms used in this paper.[24]

1. **Ground-Water Hydrology.** Ground-water hydrology is the branch of the science of hydrology that treats ground water; its occurrence and motions, its replenishment and depletion; the properties of rock that control ground-water movement and storage; and methods of investigation and utilization of ground water.
2. **Suspended Water.** Suspended water is water in the zone of aeration, including seeping water (gravity water) and stored water (capillary and pellicular water).

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[24] Unless otherwise noted, these definitions are as given in "Ground Water," by C. F. Tolman, McGraw-Hill Book Co., Inc., 1938; see also "Outline of Ground-Water Hydrology with Definitions," by O. E. Meinzer, *U. S. Geological Survey Water Supply Paper, No. 494*, 1923.

3. **Free Ground Water.** If water moves through interconnected openings of a body of pervious material unhampered by impervious confining material, it may be denominated "free ground water" moving under the control of the slope of the water table, as distinguished from "confined water" which moves in strata, conduits, or arteries under the control of the difference in head between the intake and discharge areas of the confined water body.

4. **Seepage.** According to generally accepted usage, seepage is the movement of water into or out of the ground. Movement of water into the ground from the surface down to the water table is called "influent seepage" and discharge of ground water to the surface is "effluent seepage." The terms "influent" and "effluent" are also used with reference to streams, ground water, and the water table. An "influent" or "losing stream" is feeding water underground and an "effluent" or "gaining stream" is receiving water from underground sources. "Influent ground water" or an "influent water table" is receiving water from the surface and "effluent ground water" or "water table" is discharging water to the ground surface.

5. **Cone of Pumping Depression.** A cone-shaped depression in the water table developed around a pumping well, the periphery of which (ground-water divide) delimits the ground water moving toward the well.

6. **Basin.** The term "basin" is popularly used for the large structural valleys of Western United States.

7. **Ground-Water Reservoir.** A "ground-water reservoir," the term commonly used for a free ground-water body, is not comparable to a static surface reservoir. Ground water is never static but is always percolating slowly from intake to discharge area in the subsurface reservoir.

Ground water in stream-laid sand and gravel bounded by "bed and banks" of less pervious material or impervious bedrock may be considered to function as a reservoir rather than a subsurface stream if the body of gravel is bounded above and below by bedrock narrows. Like a surface reservoir the rate of flow in a ground-water reservoir is very slow, and the body of impounded water is large.

DISCUSSION

Donald M. Baker,[25] **M. Am. Soc. C. E.** (by letter).—A number of inconsistencies and fallacies exist in many court decisions relative to ground-water occurrence and behavior. The authors are to be congratulated in bringing these fallacies into clear focus and it is hoped that the influence of the paper will be reflected in future decisions dealing with this topic.

Much of the early investigational work dealing with ground-water hydrology was occasioned by a series of court cases which arose out of the 9-yr dry cycle that occurred in Southern California around the turn of the century, and many of the legal principles laid down and classifications adopted by the courts in the western states had their beginnings in these cases.

The two decades 1920 to 1940 have seen a great accumulation of factual information upon this topic, a material increase in the knowledge of the causes and effects of the occurrence and behavior of ground water, as well as great improvement in the technique of collecting basic data relative to ground water. Blame for the failure of the courts to recognize many of these developments may lie partly with the technical experts who hesitate (because of a fear that such presentation will not "get over" with the court) to present ground-water situations in the light of the latest knowledge available regarding them, and partly with the courts themselves, who, for their knowledge of a very complicated technical subject, rely too greatly upon legal decisions written by men who, though well grounded in the law, are not familiar with the fundamental principles of ground-water hydrology. Thus a vicious circle exists, which it is hoped this paper may help to disrupt.

As stated by the authors, all ground water—that is, water occurring within completely saturated portions of the lithosphere—is in movement from the point where it enters the saturated zone to the point where it is discharged from it. If three axes, X, Y, and Z, each at right angles to the other, are taken at any point in a body of ground water, with the Z-axis being assumed to lie in the direction of movement of the ground water, the Y-axis being vertical, and the X-axis horizontal, four classifications of ground water may be made, based upon the location of the formations which restrict movement along such axes, in which:

1. The principal restriction is against movement in a downward or $-Y$ direction, such restrictions as exist against movement in a $+X$ or $-X$ direction being located a considerable distance apart;
2. Restriction is against movement in a downward direction, along the $-Y$-axis, and also against lateral movement in a $+X$ or $-X$ direction;
3. Restriction is against movement upward and downward, in a $+Y$ or $-Y$ direction, such restrictions as exist against movement in a lateral direction along the $+X$ or $-X$ direction being located a considerable distance apart; and

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[25] Cons. Engr., Los Angeles, Calif.

4. Restriction against movement is in an upward and downward direction along the $+Y$ and $-Y$ axes, and also against lateral movement in a $+X$ and $-X$ direction.

In Class 1 would fall all free ground-water bodies, commonly known as ground-water basins or reservoirs, underlain by impervious or nearly impervious formations with the upper surface of the ground-water body free to move in a vertical direction with changes in volume of the ground-water body, movement of ground water being in the direction of the slope of this upper surface or water table. Although in most instances such bodies of ground water are confined laterally, the confining formations are located far apart. A profile of the water table of such body along the X-axis would be concave or convex, or even sinuous.

Class 2 would embrace subsurface water courses, the formations restricting lateral movement being located relatively close together, in terms of the length of the body along the Z-axis; and a transverse profile of the water table, although it might be slightly concave or convex, would more closely approach a straight line.

Ground-water bodies in Class 3 would be similar in shape to those in Class 1, except that all ground water in the body would be under pressure, whereas those in Class 4 would be similar in shape to those in Class 2, but would be under pressure, the flow of ground water in Class 3 and Class 4 being in the direction of the pressure gradient.

Distinctions between ground-water bodies in the four classes are not always clearly defined, and sometimes become a matter largely of degree, particularly in regard to restrictions against lateral movement. A body having lateral restrictions 200 ft apart might be placed in Class 2, whereas if similar restrictions were a mile apart it might be placed in Class 1. Bodies in Class 1 and Class 3 have general characteristics more or less similar to surface lakes, whereas those in Class 2 and Class 4 have characteristics more akin to those of surface streams.

Many ground-water bodies may fall in one class in one portion, and another class in another. For example, ground water in a large valley may fall in Class 1 in the upper section or along the sides, and in Class 3 in the center or at lower elevations. Furthermore, a large ground-water body may be separated into a number of subdivisions, either within the same or in different classes. A body in Class 1 may be divided into a series of basins, each with a water table at different levels, separated from one another by underground dikes, faults, or ridges over which the ground water cascades to lower levels. Reductions in the cross-sectional area normal to the direction of movement will cause like subdivisions in bodies in classes 2, 3, and 4.

Legal controversies over utilization of ground water, other than those dealing with the relative rights and priorities of users from a ground-water body, usually fall into two general classes:

(*a*) The supply for persons claiming superior rights to the use of such ground water is asserted to be reduced through the diversion by the person complained against; and (*b*) The elevation of the water in the wells of persons claiming superior rights to the use of ground water is asserted to be lowered through the diversion by the party complained against.

Nearly all of the cases with which the writer is familiar are based upon claims made under Class (*a*), the plaintiff asserting that actual or threatened diversion by the defendant would diminish his supply of water, and the court making its ruling on this basis, when, as a matter of actual fact an ample supply existed to satisfy plaintiff's right, and the sole effect of defendant's diversion was to lower the elevation at which the water stood in plaintiff's well and require him to lower his pump.

There appears to be little or no discussion in ground water cases as to whether the owner of a superior right to use ground water also has the right to have the elevation of the water in his well maintained in the same manner as existed prior to diversions under inferior rights. Current knowledge and technique in ground-water hydrology allow the determination, with a reasonable degree of accuracy, of the quantity of ground water which may be diverted safely from a ground-water body over a given period of time without exceeding the supply of water which the body will receive during such time; and from such a determination the question of the probable diminution of supply necessary to satisfy superior rights may be settled.

Numerous situations exist in which increased draft upon a ground-water body will actually increase the quantity of water which may be diverted from it. Such increased draft during periods of subnormal supply enlarges the storage capacity of the ground-water body and allows more water to be impounded during periods of more than normal supply. The same situation exists when the storage capacity of a surface reservoir is increased—the increase in storage capacity allows an increase in the draft from the reservoir. Such increased draft upon ground-water bodies, however, usually results in a considerable lowering of the water level in wells used to divert from it, over extended periods of time.

If future decisions should hold that rights to divert and use water from ground-water bodies include the right to maintenance of the elevation of the water in the wells through which such water is diverted, it would be a severe blow to the interest of conservation and highest utilization of such supplies. There is a great need for clarification of this phase of ground-water law.

SAMUEL C. WIEL,[26] ESQ. (by letter).—Energetically, the authors offer the accumulating data of a new science. A hundred years ago, when law on waters was beginning, a law writer, then justifiably, wrote:[27]

> "The origin of springs and rivers is one of the mysteries of nature, which human reason has attempted in vain to develope. The inquiry how they are produced, and whence they derive their unceasing supply of water, has been variously considered, and divided the opinions of philosophers more than any other topic in natural history. The opponents in the controversy have enlisted under two leaders, De la Hire who contends that rivers are supplied from the sea, percolated through the pores of the

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[26] Attorney at Law, San Francisco, Calif.

[27] "Watercourses," by Joseph K. Angell, 1st Ed. (1824).

> earth, and Doctor Halley who has endeavoured to demonstrate that the clouds alone are sufficient for the supply. Each has had recourse to mathematical calculations, and the results of each have been equally unsatisfactory."

Today, the passages from legal discussions, cited by the authors, show legal cognizance of such developments of hydrology as catchment basins, underground reservoirs, water levels, underground streams, permeability, underflow, and saturation.

This legal hospitality to the data of hydrology indicates more to come as hydrology is patiently presented. The use of ground-water contours, ground-water mounds, and ground-water troughs which the authors expound may be expected to come into legal discussions. These graphic methods do not produce the conditions which they describe, any more than security-market graphs make security prices; but they are evidently an advanced medium of description which legal authorities can, and doubtless will, learn from hydrologists. It is fair to assume that some patience will be required, however.

"Hydrology," A. F. Meyer wrote in 1928,[28] "is essentially a new science" and "our present knowledge of the subject is indeed fragmentary and incomplete." The first general treatise published in English devoted primarily to the part of hydrology dealing with ground water, Professor Tolman has noted, appeared in 1937.[29] Naturally, in this newness there are conflicts among hydrologists. Mr. Meinzer[30] writes of localities where "some of the water may be virtually stationary in synclinal troughs or encased lenses"; the authors state (see heading "Percolating Waters: Subsurface Reservoir") that "ground water never occurs as a stationary water body." Baker and Conkling[31] declare that the proper conception of ground water in alluvium is that of "water in a reservoir"; the authors maintain that such comparison is not justifiable. Elsewhere, Professor Tolman has stated that "No water table can exist in impervious material";[32] Meinzer intimates that in most places no material is absolutely impervious, unless, perhaps, he means, at considerable depth. ("Even the compact igneous and metamorphic rocks and the dense clay and shale formations have in most places become sufficiently permeable by weathering near the surface to have a water table."[33]) Naturally, a new science's debates slow its reception.

Law also has its factors for patience. Not all rests with courts. The legislatures share in setting up legal rules. Citing the law's differentiation of "subterranean streams flowing through known and definite channels" from other underground water, the authors (see "Subsurface Water Course") conclude that "courts refuse to accept scientific evidence indicating natural conditions."

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[28] "Hydrology," by Adolph F. Meyer, p. 1.

[29] "Ground Water," by C. F. Tolman, McGraw-Hill Book Co., Inc., 1937, page vii.

[30] "Groundwater in the United States," by O. E. Meinzer, *U. S. Geological Survey Water Supply Paper, No. 836-D*, 1939, p. 182.

[31] "Water Supply and Utilization," by Donald M. Baker and Harold Conkling, 1930, p. 346.

[32] "Ground Water," by C. F. Tolman, McGraw-Hill Book Co., Inc., 1937, p. 225.

[33] "Groundwater in the United States," by O. E. Meinzer, *U. S. Geological Survey Water Supply Paper, No. 836-D*, 1939, p. 174.

Although it originated in courts, the differentiation is now frequently prescribed by legislation.[34]

Perhaps the most important moderator on the rate of reception of hydrology into law is the fact that what takes place in courts about water never occurs in the abstract as in the classroom or office, but is part of a drama of opposing sides. Unless the engineer will include human nature in his conception of it, his combination-rate for hydrology and law is certain to be too high.

In the classroom or office one is unopposed. In court, nothing can be determined against persons who have not been brought in as opponents, which leads in practice to a marked and often decisive curb compared to the expansive latitude in the classroom or office. Then, among those brought in, nothing can be determined outside the issues made by preliminary statements and counter-statements of what is intended to be presented. These may be formed from such considerations as length of time the controversy has lasted, representations that one party has made to the other, the presence or absence of writings, and so on in endless variety that may be more vital to the parties than physical data.

The trial of the issues, after they are formed, is full of adventure. As much as one is unopposed in the classroom or office, in court little that is asserted will be accepted that has not been proved; and for hydrology this means proof by witnesses. Obviously, what a judge may assume to be true, by judicial notice without witnesses, must be kept very limited. "If there is any doubt whatever, either as to the fact itself or as to its being a matter of common knowledge, evidence should be required."[35]

An illustration occurs when one bases a claim upon existence of a known condition of a particular kind, such as a known and defined underground stream, already noted, which the authors discuss. When a litigant contends for a superiority on that score there is no escaping that the facts are in no two places the same. "The point at which an underground stream merges into a body of percolating water is sometimes as difficult to determine as the point at which a river entering a lake ceases to be a river and becomes a lake, and depends upon the facts of each case."[36] It is evidently like the miners' contentions of knowing that their veins of ore run underground into a property where they are being extracted by another. When denying a claim based upon existence of a known and defined underground stream, the ground water is called "diffused" only in the sense of "undefined" by the evidence. The authors seem to be a little too inclined to assume that this bars parties from proving a motion where they can. The writer believes that what it requires is only that the motion and other hydrological characteristics and environment be proved by evidence in each case according to the locality and instance. Property may not be awarded on "no more than a guess or a wish."[37] Would it be agreeable to hydrologists to have a judge conjecture on a locality's hydrology by judicial notice without any witness testifying?

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[34] California Water Commission Act, Sec. 42; Arizona legislation reviewed in Maricopa County Municipal Water Conservation District vs. Southwest Cotton Co., 4 Pac. (2) 369, 375–377 (1931), 7 Pac. (2) 254 (1932); Oklahoma legislation reviewed in Canada vs. City of Shawnee, 64 Pac. (2) 694, 698 (1936).

[35] Chowchilla Farms vs. Martin (1933), 219 Cal. 1, 25 Pac. (2) 435, 439.

[36] "Water Supply and Utilization," by Donald M. Baker and Harold Conkling, 1930, p. 321. Accord, San Bernardino vs. Riverside, 186 Cal. 7, 198 Pac. 784 (1921).

[37] Evans vs. City of Seattle (1935), 182 Wash. 450, 47 Pac. (2) 984, 986.

Failure to produce evidence thus reflects less on the court than on the litigants. However, it is not necessarily a reflection because comparatively few water suits are between properties of a value to justify much expense for evidence. When expert evidence is used, moreover, the expert having an effective personality is likely to be preferred, the same as in selecting between possible witnesses to be offered on any other matter. He may then carry something that hydrologists who later read the decision will dislike.

That is not all, however. The other side will have its contrary expert. In the classroom or office, hydrology can be stated abstractly and with indifference; but called to take up a quarrel for one side the lecturer or engineer, like the attorney, begins to warm up a sympathy for the side calling him; his reasoning powers and learning will in turn warm up his sympathy; he will deem it a merit to detect anything new in his side's favor; then he believes honestly that it is the only right side and is as convinced of this as is that side's attorney. When the opponent's expert appears with a contradiction, the first expert's zeal for his client passes into defense of himself.

It has been said:[38]

> "These witnesses ought, perhaps, to be selected by the Court, and should be impartial as well as learned and skillful. A contrary practice, however, is now probably too well established to allow the more salutary rule to be enforced, but it must be painfully evident to every practitioner that these witnesses are generally but adroit advocates of the theory upon which the party calling them relies, rather than impartial experts, upon whose superior judgment and learning the jury can safely rely. Even men of the highest character and integrity are apt to be prejudiced in favor of the party by whom they are employed. And, as a matter of course, no expert is called until the party calling him is assured that his opinion will be favorable."

Being said, as this was, in 1870, it shows an early instance of the contention that expert witnesses should be called by the court instead of by the parties. This contention has provoked some discussion on legal lines, but perhaps more on the line that might be called "socialized hydrology," in analogy to the now-current discussion over "socialized medicine." Some approach thereto may perhaps be seen in the authority sometimes given to the courts to refer water cases to the State Water Administration for report. Such authority exists in a number of states.

In all this, the parties, advocates, witnesses, and at times jurors, are men of all degrees of mental and social equipment, temperament, and predispositions. Men are influenced more by their interest, anger, hope, fear, and the like than by rules of law. So is the judge; but, to the degree that he is a good judge, he will be silent and remain skeptical of all the witnesses together until the decision is made, under his oath of impartiality. On appeal, this scene is preserved in the printed record of the trial, with additional plotting and counterplotting to move the judges in the appellate arguments. It is said in a not uncommon instance:[39]

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[38] Grigsby vs. Clear Lake Water Co. (1870), 40 Cal. 396, 405.

[39] Tulare Irrigation District vs. Lindsay-Strathmore Irrigation District (1935), 3 Cal. (2) 489, 45 Pac. (2) 972, 1009.

> "From reading the expert testimony presented by appellant, one would be led to believe that the Kaweah delta is rapidly becoming either a swamp or an alkali waste, while the experts for respondents would have us believe that the delta is a veritable Garden of Eden."

The courts necessarily sometimes must take refuge simply in deciding against the party who has the burden of proof.

Hydrology in litigation must survive this very adventurous human journey the same as other litigated matter, before its entry into decision is possible. At least some criticism of the courts must be discounted as the same disappointment of a losing side that is inseparable from a human drama wherein chance of loss is inherent, whatever the subject.

If hydrology's entry into judicial decision is therefore slow or sometimes imperfect, it is fair to assume that its acceleration waits on practical considerations. Among these may be suggested acquainting the smaller litigants with the benefit that hydrologists, as witnesses, can be to them. Such witnesses might also be made available to them in some way at affordable expense. As it becomes systematized by such work as that of Professor Tolman, hydrology will also progress, as a matter of course, toward precision that will make faulty experting more easily refuted. But such considerations will evidently need a little time.

Although the authors emphasize what hydrology has compiled, and the speed with which they would like to see it appear in legal depositions, the writer wishes to emphasize rather the converse service that hydrology can perform. By affirming the great scope of what must continue to be indeterminate, hydrology can clarify the legal consequences from some popular misassumptions.

Much of the unknown, that prevailed when the law of the subject began, will not be eradicated for some time to come, even with the best patience.

Mr. Meinzer—credited by the authors with leadership of the subject—is insistent upon this view. He says the facts or truths involved in ground-water hydrology "are known in only small fragments" and he "has become impressed with the large and difficult field that the subject covers and the variety and complexity of the concepts that it involves."[40] As recently as 1939 he has spoken of "almost sensational revelations, showing that both water tables and artesian pressure are constantly fluctuating in complicated fashion in response to a variety of impinging forces."[41]

Other commentators likewise[42] are usually ready to affirm this residual indeterminateness. Geologists have testified in court that a certain ground-water occurrence was as much a surprise to them as to others.[43] The courts sometimes find that the evidence presented to them discloses that it is difficult even for the most experienced geologists to determine, definitely, ground-water's source, course, or destination under certain circumstances;[44] that how long it will take for a particular ground-water change to come about "we doubt if anybody knows,"[45] or "probably no one knows."[46]

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[40] "Outline of Ground-Water Hydrology," by O. E. Meinzer, *U. S. Geological Survey Water Supply Paper, No. 494*, 1923, pp. 1, 2.

[41] "Groundwater in the United States," by O. E. Meinzer, *U. S. Geological Survey Water Supply Paper, No. 836-D*, 1939, p. 212.

[42] Report of Special Advisory Committee submitted to the Secretary of the Interior, May 26, 1936, by National Resources Committee, p. 9.

[43] O'Leary vs. Herbert (1936), 5 Cal. (2) 416, 55 Pac. (2) 834, and cf. Joldersma vs. Muskegon etc. Co. (1938), 286 Mich. 520, 282 Northwestern 229.

The unknown affords opportunity for unrealities to get belief. Disregard of the residual indeterminateness of the subject readily leads to embracing some form of legal separation of rights by the process of dividing-up water resources into independent parts like land lots.

One form thinks of dividing the water-body or "corpus" into detached masses by vertically projected land-boundary lines.

Will this stand hydrological analysis? How water resources of whatever kind, retaining so much that is indeterminate, can yield such a separation into parts, is difficult to understand. Water resources are compounded of runoff, seepage, evaporation, and plant transpiration, controlled by the wind, rain, heat of the sun, and forces in the depths of the earth, all of which are indivisibles. The writer interprets the paper to convey, emphatically, that all parts of a given water body are inextricably inter-related.

Particularly, they are in motion—the authors insist, always in motion (see heading, "Percolating Waters: Sub-Surface Reservoir")—and even a surface stream has ground water entering or leaving the bed as an important feature. In this connection, Clarence Johnston, M. Am. Soc. C. E., has stated:[47]

> "Streams and lakes are not like land, in so far as private possession is concerned. Land can be measured and privately controlled. Water is constantly shifting, and the supply changes every day."

Motion gives streams the unique character of being a perpetual renewal of transient contents. Unification of a stream's coming and going—and it is more or less the same of all other water resources—exists only subjectively in the observer's eye. Just so the apparent unity which the succession of impressions on the film of a motion picture has on the screen is only a subjective unity. The apparent single objectiveness is the same visual illusion in both instances; and if this is realized there is nothing that answers to a determinate block of water for land-boundary lines to divide into the lots supposed.

It brings to mind a case in which:[48]

> "The discovery of a statue of Pompey, ten feet in length, was the occasion of a lawsuit. It had been found under a partition wall; the equitable judge had pronounced that the head should be separated from the body to satisfy the claims of the contiguous owners; and the sentence would have been executed if the intercession of a Cardinal and the liberality of a Pope had not rescued the Roman hero from the hands of his barbarous countrymen!"

Popular discussion, and law books too, show some unsuspecting efforts to adjust legal rights by dividing the water "corpus" by the geometrical process mentioned. It ought not to be too much to expect of hydrology that it stand against that fanciedly simple legal solution, often entertained by the public for allotting water resources, and by some legal and engineering circles.

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[44] Silver King etc. Co. vs. Sutton (1934), 85 Utah 297, 39 Pac. (2) 682, 690.

[45] Berry vs. Shell etc. Co. (1934), 140 Kan. 94, 33 Pac. (2) 953.

[46] Comar Oil Co. vs. Blagden (1934), 169 Okla. 78, 35 Pac. (2) 954, 955.

[47] *Transactions*, Am. Soc. C. E., Vol. LXXVI, December, 1913, p. 683.

[48] "Roman Empire," by Edward Gibbon, Ch. LXXI, Everyman Ed., Vol. VI, p. 567.

One should not be deemed to be going too far in suggesting also that hydrology should stand against the comparable popular solution that professes to divide water resources into legal parts in another way. This assumes to segregate parts from the remainder mathematically according to priority in time of breaking off the part—that is, "prior appropriation." In searching the expressions of the past one finds that natural water supplies have been frequently treated in law as though they were bank accounts as a "corpus" of "appropriable water" to be drawn from in certain amounts by a simple process of subtraction.

The multifarious qualifications that have come to surround that extended effort show how this supposed mathematical simplicity of dividing-up the visual illusion is beset with the same hydrologic indeterminates; for example:[49]

> "Administration of underground water involves gathering and studying data. The work is in the office, and decision is not so easy. The cost of obtaining sufficient information to provide a basis for intelligent decision is considerable, and this cost would not be justified where economic conditions do not permit * * *."

In some instances, it has had to involve measuring absorptive capacity of all stream beds in a large county.[50] Elsewhere, constant public records of thousands of well readings are being introduced. Indeed, courts that are careful sometimes find water conditions, whether underground or on the surface, so indeterminate that experiment is compelled. They sometimes find it necessary to reserve jurisdiction so that they may modify their orders, or they find it necessary to appoint commissioners to keep in touch with the obscure or changing situations.

Whoever has charge of legal adjustment on the conception of mathematical division of water resources by priority of "appropriation" of parts, finds himself, sooner or later, in difficulties. It has been one of litigation's most fruitful sources. He comes, through trial and error, to suspecting whether water-division formulas, be they geometrical or mathematical, apparently so simple, are searching in a dark room for a black hat that is not there. He confesses the need for a considerable degree of discretion to bring, out of indeterminate physical circumstances, the best human result that he can in his own way. He needs a considerable degree of latitude to leave formularizing, and apply such disposition as he believes to be "reasonable use" on the part of the one water claimant toward the other; that legal position to which the discussed paper refers by saying (and the writer believes, correctly) that although several states of the West have extended rights by prior "appropriation" to ground water, the general trend "seems to be toward application of equal rights" (see "Part II.—Analysis of a Few Rules of Law").

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[49] "Administrative Control of Underground Water: Physical and Legal Aspects," by Harold Conkling, *Transactions*, Am. Soc. C. E., Vol. 102 (1937), pp. 753, 778-779; also "San Gabriel Investigation," by Harold Conkling, *Bulletin No. 7*, Division of Water Rights, Calif. Dept. of Public Works, p. 14 *et seq. passim*.

[50] "Ground Water," by C. F. Tolman, 1937, p. 179.

Since appropriation signifies mathematical division into parts by priorities, the trend from it to equality of standing based upon discretionary "reasonable use" of water claimants toward each other is evidence that hope of dividing-up water resources, whether geometrically like land lots or mathematically by priorities in time, however easy of imagining and of verbally expressing and enacting, is lacking in something under the water authority's experience.[47]

The law is increasingly coming to rely upon how a situation which the water authority faces impresses his good sense. If this is unfortunate—and perhaps it is—the writer ventures to think that it is nonetheless unavoidable, with loyalty to reality.[51]

The discretionary authority spoken of (which is to say what is "reasonable") has been, at common law, the courts; but current practice is preferring to put it by legislation into administrative officers in the first instance. In the West, particularly, there are extensive State Water Administrations which, while initiated to achieve mathematical divisions by priorities, practice large discretion in the field. As time advances, the writer believes there will follow less occasion for the terms "riparians" and "appropriators" that now divide the subject.

Of course, few in the United States would contemplate that discretion should so grow as to become dictatorial power. To prevent it becoming so, it is to be hoped that a hearing in court, dedicated as the courts are to impartiality, will be preserved to complaints against abuse of the discretion. However, that is too large a subject, throughout the country, for discussion herein. The better the water administrators are posted in hydrology the fewer, one may hope, will be the abuses.

The writer would venture a final comment upon cooperation of hydrology and law. It is long-standing law everywhere that when water disturbances are made for public use, private claimants of whatever origin may be compelled to yield their water rights by conversion to a money equivalent measured by the damage suffered, what is called condemnation under power of eminent domain. Under the usual statutes, this needs only a hearing to establish the public necessity and the amount of the damage or, if such be the fact, the absence of damage—followed, of course, by payment of what damage is proved. The instances in which public use meets legal hindrance usually find the public use insisting upon proceeding with its work while leaving the question of the amount of damage, or of the absence thereof, unheard and untried. Public users sometimes feel that when they say they are doing no damage the law must take their word for it.[52] There is nothing special to water law when that attitude is disallowed; nor any hindrance to public users by whom trial of damage is willingly entertained. I have ventured to suggest elsewhere that some hydrological principles bearing upon the amount of monetary damage can be formulated.[53]

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[51] "Fifty Years of Water Law," by Samuel C. Wiel (1936), 50 *Harvard Law Review* 252, 16 *Oregon Law Review* 203.

[52] "San Francisco's Riparian Water Case," by Samuel C. Wiel, 13 *Southern California Law Review* 204 Univ. of Southern California, Los Angeles, Calif., January, 1940).

[53] "The Need of Unified Law for Surface and Underground Water," by Samuel C. Wiel, 2 *Southern California Law Review* 358, 1929.

Hyde Forbes,[54] M. Am. Soc. C. E. (by letter).—The only purpose the writer has in discussing this paper is to "carry a banner" for the legal profession and courts whom the authors feel are inadequately informed and thus hold "erroneous concepts." Many of the concepts presented by the authors are not in accord with those developed by the writer through his experience and observation; but this need not be considered germane to the object of the paper which is stated to be a discussion of "the problems involved in court interpretation of the type of ground-water occurrence according to the present classification of subsurface water for legal purposes."

The writer has found attorneys quick to understand and appreciate geologic conditions and hydrologic phenomena and able to express these conditions in the clear and concise language necessary in all legal proceedings. Furthermore, he has found the courts quite able to follow the reasoning of expert witnesses in these matters when that reasoning was kept within the bounds of fact and not carried into the realm of, what attorneys have termed, "scientific speculation."

The opinions of courts cited by the authors apply to geologic and hydrologic conditions familiar to the writer in many cases. In such cases the opinions are most applicable to the conditions obtaining and are written in a concise manner, clear in intent and meaning. The principal item of supposed conflict between legal application and hydrologic principle, pointed out by the authors, may be one of definition. About 1923 or earlier, Mr. Meinzer canvassed the field, and those active in the field, of ground-water hydrology, in an effort to organize the facts and principles pertinent to the science, and to give the concepts involved a definition, and provide a nomenclature. This was exceedingly well accomplished and was presented during 1924.[5] The writer has found no reason to amplify or change the definition or nomenclature set up therein in presenting a great variety of ground-water conditions to the laymen or lawyers. The passage of water through interstices is the scientific meaning of the word "percolation" and is that accepted by the legal profession. Percolation is so defined in the dictionary. The legal definition applied to subsurface streams has nothing to do with the definition of percolating water in that a subsurface stream has a physical occurrence as a definite water course located beneath the earth's surface between well-defined limits or banks, through which water flows or percolates in contradistinction to a condition of no such flow or percolation through the banks.

In some western states the rule of appropriation extends to ground water moving in a water course having well-defined bed and banks confining the water within circumscribed limits. Undoubtedly that rule means just what it says and is not applicable to the body of ground water percolating widely beneath great areas of land. Otherwise the appropriator who first drilled wells could draw the water from beneath all of the lands and deprive his neighbors of a valuable property right to the benefit of distant lands or regardless of reasonable use. The Arizona case cited by the authors[20] is one in point. The authors are correct when they state that "the trial court accepted the geological evidence as conclusive" as to the occurrence of subsurface streams; but they neglect to state that the evidence accepted was that presented by one party to the action—

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[54] Cons. Engr. and Geologist, San Francisco, Calif.

the client of one of the authors—and that the opposition presented a mass of geologic and hydrologic evidence of the fact that the occurrence of ground water in the Salt River and Aqua Fria valleys of Arizona was of diffused percolating character. The Supreme Court of Arizona wrote its decision, parts of which are cited, after a review of all of the evidence presented by both parties. It can scarcely be claimed that either the lower or the supreme courts refused "to accept scientific evidence indicating natural conditions"; nor that its decision "may not be well founded." It should not be said that the court established "a ruling based on its interpretation of existing geologic and hydrologic conditions." The Supreme Court studied and weighed the evidence submitted, made its findings of fact, and applied a rule of law.

The authors state (see heading "Percolating Waters"): "The most curious concept expressed in legal literature is that of 'diffused percolations * * *.' " Experience in investigating the chemical character and temperatures of ground water indicates that there is a considerable diffusion of waters that are percolating underground. Waters from many sources mingle and become diffused throughout ground-water basins. All ground water in the process of percolation is well described as "vagrant, wandering drops moved by gravity in any and every direction along the line of least resistance," when the distinction is sought to be made between widespread percolation and the subflow of a stream.

The statement, "The legal dictum that a surface stream is 'supported' by subflow is only applicable under effluent conditions," is not correct. The legal dictum, if there is one, consists of the fact that the subflow is part of the surface stream and is supplied therefrom. The subflow acts as a "support" of the surface stream in the sense that seepage of water from the surface stream is at a minimum with the maintenance of the subflow. If the subflow is removed by diversion under the water-table conditions pictured in Fig. 1, 2, or 4 the condition of Fig. 3 results with a consequent increase in seepage loss from, and a lessening of flow in, the surface stream.

In City of San Bernardino vs. City of Riverside[23] the concept of "support" cited by the authors exactly fits the physical conditions held requisite by the authors, as the streams in question were effluent streams. The citation applies to no other condition. However, in Lemm vs. Rutherford[22] the court found that the seepage from a ditch saturated the material over which the ditch flowed, and the ground-water mound so created acted as a "support" for the surface flow in that it materially reduced subsequent seepage loss therefrom. The taking of ground water from the immediate vicinity of the ditch was found to have the effect of creating an artificial draft upon the surface flow. Similar conditions attach to the other cases cited wherein artificial draft draws down the ground-water level, and in this manner "weakens and injures the natural bed."

The criticism of lawyers on the basis that "they do not generally appreciate the fact that stream flow occurs over non-saturated gravels through which water seeps from the surface to the water-table" certainly is not applicable to the lawyers met with in the writer's experience. The assertion ascribed to them would have more general application if the authors would change the words "before there can be any surface flow" to "before there can be no diminution of surface flow." As seepage from the stream builds a mound on the ground-water surface toward stream-bed elevation (as illustrated by the authors) the rate of seepage from the stream is reduced when the partly saturated capillary fringe reaches the stream bed, and seepage ceases when the ground-water elevation corresponds to stream-bed elevation. The interstitial water then acts as a "support" for the surface stream. There is nothing erroneous in the assumption that the conditions will be reversed if the ground-water elevations were to be lowered by draft upon ground water. The time affected and the measure of the effect that ground-water draft has upon surface flow is a matter that is determinable for each specific set of conditions. Some of the positive statements in that regard made by the authors toward the end of the paper are not warranted.

It is not clear what "hydraulic laws" the authors refer to in the statement ("Part I.—Occurrence and Movements of Water Underground: Effects on Surface Flow of Pumping from Influent and Effluent Subflow"): "It has been stated in legal proceedings that the ground-water level responds to pumping in the same way as the water level of a surface reservoir, which, of course, is not in agreement with hydraulic laws." Necessarily a body of ground water must be held in some container, which container corresponds to a reservoir for surface water. Any diminution in the quantity of ground water in such a basin will lower the level of the ground water over the basin. The only difference between ground-water basins and surface reservoirs in this regard is in the lag in time before a widespread effect is registered due to the frictional resistance of the materials containing the water in underground reservoirs retarding the immediate adjustment of levels.

The authors present ground-water contours as "indicative," whereas they should be referred to as "illustrative." Lawyers and the courts rightfully find it difficult to distinguish between fact and opinion when the hydrologist is not careful to make the distinction himself. It is only one step further in their confusion to assume opinion to be "unproved theory." Contour lines depicting lines of equal elevation of the ground-water surface or water table are interpolations between observations of the water surface elevation in wells or test holes scattered over an area. They are unlike ground surface contours in that they are an expression of an opinion rather than a fact. In mapping surface topography the details are presented to the eye and can be readily expressed in the interpolation made between points of determined elevation. Ground water is hidden and contour lines of its surface are necessarily the result of interpolations between measured points, expressing the judgment of the delineator based upon his experience. In the presentation of such lines to a court, one does so only as an expression of opinion based upon all determinate facts. In themselves, they are not conclusive and cannot be considered separate from the testimony and qualifications of the delineator.

The statement of the authors, "the notion that ground water must be concentrated in a restricted subsurface channel to constitute a supply sufficient for industrial or agricultural use * * * accepted by some lawyers," must have very limited application. Attorneys familiar with western water law are well informed upon the occurrence of and rights in widespread percolating water and the waters of ground-water basins. The legal decisions take cognizance of the physical facts relative to "the particular conditions existing in the area in controversy" and apply the law to establish the rights of the parties. Possibly there is room for improvement in the law applied to ground water as it has been developed in the west through a succession of court decisions. Possibly enactment of legislation might provide that property rights in ground water shall be administered more effectually than through trying each case on its merits in light of the conditions unique to each locality in controversy. However, as the great variety of ground-water occurrence and conditions is hidden below the surface of the ground, differences of opinion will arise between those whose duty it is to inform the courts or administrative body upon the nature of the conditions and effects of proposed or accomplished actions if such conditions and actions are the subject of controversy.

Engineers and geologists who, through experience, have gained a knowledge of ground-water hydrology can clarify seeming misconceptions and inconsistencies if, when controversies arise, they will recommend to their clients and attorneys that they be allowed to make investigations jointly with representatives of the other party to the controversy to the end that questions of fact will be settled through stipulation. The lawyers and the courts will have no trouble in applying the law to the facts.

The legal profession and the courts use a phraseology which may not express a "concept" as it exists in the mind of the hydrologist but is clear and precise in intent and meaning to the legal and lay mind. The ground-water hydrologist should use simple words in common usage and defined in the dictionary to express his concepts in his dealings with the legal profession. The "coinage" of new terms and phrases in court testimony as well as other discussion creates confusion because the user must define and qualify each term upon each occasion of use. The difficulty that some expert witnesses have in making their ideas known to attorneys and the courts is that they do not present their reasoning in a manner that can be followed by the lay mind. All geologic and hydrologic phenomena can be presented in a simple manner, identified with ever present examples of familiar processes. The correctness of scientific reasoning can often be checked by the fact that it presents, in the words of the Supreme Court of Arizona,[20] "clear and convincing proof to the satisfaction of a reasonable man."

Ronald B. Harris,[55] **Esq.** (by letter).—Although he is a lawyer, the writer recognizes the justification of the criticism to the fact that certain court decisions have expressed erroneous concepts of the nature of ground water.

The courts are the finders of facts which they cull from the evidence presented to them. It is the duty of the courts to distinguish between that evidence which is true and that which is false. Generally speaking, facts are presented by witnesses and where the subject matter calls for special knowledge, only those persons who can qualify as having special knowledge in that field are entitled to express opinions. If the decisions of the courts express erroneous concepts of the nature of ground water, then such decisions reflect one of two conditions: either (1) that at the time of the trial of the action the nature of ground water was not fully developed and known, except by a few pioneers in that field, and hence expressed the then deficient popular knowledge concerning the same; or (2) that the witnesses testifying in that particular case were not qualified.

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[55] Harris, Willey & Harris, Attorneys at Law, Fresno, Calif.

Of course, it is true that in all fields of learning there are charlatans who impose themselves upon the courts as experts, or, although experts, knowingly betray their profession; and their testimony, often being quite plausible, sometimes influences the courts' decisions.

However, in the matter presented in the paper, the decisions reflect the then popular concept of ground water. The need of accurate knowledge concerning ground water did not become pressing until the introduction of pumping. This did not occur until, within certain areas, surface streams were exhausted and the quest for water went underground. When pumping exceeded replenishment of the ground water, with the resultant lowering of the water table, water users affected were willing to finance the costs of investigating and determining the nature of ground water—first for the purpose of preserving its level and, second, for determining rights to its use.

The leading case in California on rights to ground water is Katz vs. Walkinshaw,[56] which applied the doctrine of correlative use to percolating water. This doctrine, of course, was already in use as between riparian users as a class. It is the writer's opinion that this doctrine is being extended and, in time, will be the universal doctrine for measuring the rights of all users of water, regardless of whether their rights arise as riparian owners, as appropriators from surface supply, or as users of ground water. In other words, all water users within a drainage area are joint users of all the waters that occur in that area and each user's rights will be correlated with each other user's rights, no matter from whence their source of supply. The necessity of making nice distinctions in classifications as to the source of supply being natural stream flow, flood water, subsurface streams, percolating water, etc., will be eliminated gradually because the measure of the user's right will be limited to that use of all the waters that are available to him, regardless of the source, which will result in the most practical and economic use of all the waters in relation to all of the users. Priorities will still be recognized but will be correlated to all other priorities in their order not necessarily as to specific sources of supply but in consideration of all sources of supply.[57]

The application of this doctrine, no doubt, will be extended to each area if, as and when, due to drought or other factors, the demand exceeds an uneconomic use of the water supply in such area. In making the greatest economic use of the waters in an area, ground water must be included.

The prevailing thought now is to prevent waters that could be put to beneficial use from escaping to the sea. The methods of accomplishing this are by stream reservoirs, by percolation basins, and by retardation by ground storage caused by irrigation. (This process is "retardation" because, although below the surface of the ground, the water is not static. It generally moves slowly in the direction of the topographical slope, and at the lower fringe of the alluvial cones or plains would seep into natural water courses and thence to the sea.)

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[56] 141 Cal. 116 (1903).

[57] Peabody vs. City of Vallejo, 2 Cal. (2) 351 (1935); City of Lodi vs. East Bay Municipal Utility District, 7 Cal. (2) 316 (1936); Hillside Water Co. vs. City of Los Angeles, 10 Cal. (2) 677 (1938); and Subterranean and Percolating Waters, 190 A. L. R. 395 (1937).

Throughout the western states, through federal aid, millions of acres of arid lands are being brought under artificial irrigation. In doing this, great changes will be made in the ground-water tables. An example of the importance of ground water in connection with irrigation is the Kings River area in California. In this area there are about 800,000 acres of land partly irrigated from Kings River. This irrigation creates a high ground-water table which is used as a supplemental supply for irrigation. About 12,000 pumping plants utilize this source of supply with a charge for electrical energy of between $1,500,000 to $2,000,000 per annum. This ground-water table becomes of paramount importance in those years when the seasonal runoff of Kings River is wholly insufficient to meet irrigation needs.

The question may well be asked, what relevancy has the foregoing to the paper under discussion? It has this relevancy: The paper demonstrated the lack of disseminated knowledge as to the nature of ground water. The trend in the law following the development of large areas of arid lands through irrigation, with its accompanying creation of new and changed water tables, requires thorough and accurate knowledge of ground water, not only in settling disputes as to the rights to use of such ground water, but in its control for the beneficial and most economical use, and in the preservation of lower lying lands from destruction by too high a water table. For this courts and water users must look to irrigation engineers. The fountainhead for their learning is the universities. Are the universities fulfilling this requirement in the education of their engineers who desire to specialize in irrigation? Are these engineers being qualified as experts in ground water? It is the understanding of the writer that, generally, they are not.

It is the writer's thought that the place to correct the misconceptions of the nature and characteristics of ground water is at the source of learning, and that is, at the universities.

EDWARD F. TREADWELL,[58] ESQ. (by letter).—The writer does not presume to discuss the principles of hydrology set forth in this paper. Of course, both lawyers and courts must sometimes consider and discuss such principles, and they recognize the difficulty of speaking on that subject with such accuracy as meets the exacting demands of science or scientists. The paper may be of assistance in aiding the legal profession in understanding the principles of hydrology applicable to underground water and in stating them with accuracy.

At least half of the paper is devoted to criticisms of judicial decisions in cases involving some of those principles. As to that portion of the paper one may presume to offer some comment.

Practically every case cited is attacked as being in some way in violation of the positive rules of hydrology developed by the paper. This probably indicates that the scientist has as much difficulty in understanding a judicial decision as the lawyer has in stating a scientific principle. Such a paper would be of much more value if it had been prepared in cooperation with a lawyer. The writer disagrees entirely with the criticisms made of the decisions in question.

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[58] Treadwell & Laughlin, San Francisco, Calif.

The first case criticized is Maricopa County Municipal Water Conservation District No. 1 vs. Southwest Cotton Company[20] involving the underground water of Salt River Valley. In that case the trial court apparently held that the plaintiff had certain rights on the "theory" that the underground water was water of an underground watercourse. The authors of the paper seem to complain that the trial judge properly held conclusive the "geological evidence," and that the Supreme Court disregarded the geological or scientific evidence; and they lament that "if courts refuse to accept scientific evidence indicating natural conditions their decisions may not be well founded." As a matter of fact, the Supreme Court did not refuse to follow scientific "evidence," but held that there was nothing more than scientific or geological theory of channels somewhere in Salt River Valley, but no evidence as to where these underground channels were located. It even accepted the "theory" of such channels, but still held there was no "evidence" of their location. This does not seem to be subject to criticism. Differences arise even as to the existence of surface streams, and different courts have come to different conclusions on the same evidence. It is not strange that different conclusions are reached as to underground channels. Geological principles may be as positive as the authors indicate; still the courts need not follow every witness that attempts to establish the "facts" to which such principles are to be applied. A principle of geology can no more be applied than can a principle of law, until the facts are determined. The court does not violate the principle in either case because it refuses to apply it on the ground that the necessary facts have not been established.

The next case criticized is Vineland Irrigation District vs. Azusa Irrigation Company.[21] In this case the Supreme Court was not stating what occurs under certain conditions, but what the trial court found actually occurred. The trial court found, as a fact, that the taking of the water creates an artificial draft upon the surface flow of the stream, draws down a part of it, and weakens and injures the natural bed of the stream, and tends to interrupt and carry away the surface flow. The trial court also found that the stream "saturated" the gravelly bed of the stream. Still the authors of the paper misconstrue this decision into holding that an influent flow generally supports the surface flow. In fact, the condition involved was exactly what the authors define as "an influent subflow, with ground-water mound in contact with surface flow." This must be so if the entire bed was saturated. The decision, therefore, seems proper.

The next case apparently criticized is Lemm vs. Rutherford.[22] In that case the right to construct a sump into which the seepage from a nearby ditch was collected and from which it was taken was involved. The court was careful to point out that there was no independent body of subsurface water. The only source of underground water was the seepage of the water from the ditch. The intimation of the paper would seem to be that the seepage in no way supported the water in the ditch and, therefore, that the landowners should be permitted to take it. Of course, all earthen ditches seep and the amount of the seepage is due to the porosity of the material through which they pass. Even a porous material is some support for the water in the ditch; otherwise all the water would seep from the ditch. The water that fills the pores of the material also acts as a support. If that support is taken away by a trench or sump near the ditch from which trench or sump water is pumped, then there would seem to be room for the claim that such artificial works induced seepage and thus took away the right of natural support. The writer can see no reason why this could not occur.

The next case criticized is City of San Bernardino vs. City of Riverside.[23] Everything stated by the court in that case is based upon the assumption that the stream runs over porous material saturated with water, and that the underflow water supports the stream, either by upward or lateral pressure, or feeds it directly. The authors misinterpret this into the "concept that 'support' is generally afforded the surface stream by subflow," which concept they state is erroneous. Here again they overlook the finding that the material was "saturated," which brings the subsurface water in contact with the surface stream, which condition the authors themselves admit does support the stream.

The writer is entirely unable to understand wherein any of these cases violates any positive rule of hydrology, even if he were to admit the correctness of all the rules stated by the authors. As stated, the writer does not presume to question any of those rules, but he would like to read further evidence to support the positive rule announced by the authors that as long as there is a column of unsaturated material between the surface stream and the underground water table, a lowering of the water table will not affect the surface stream. May the writer be so bold as to suggest that the water table is at all times supplying this column with water, by capillarity, for several feet above the water table. The lowering of this table reduces that supply, and this leaves just that many more voids to be filled by the seepage from the surface stream. Irrespective of whether or not such facts would make the taking of water from the underground body unlawful, the writer believes that these considerations might tend to weaken the absolute rule stated by the authors.

O. E. MEINZER,[59] ESQ. (by letter).—In showing clearly the confused and erroneous character of many of the legal concepts relating to ground waters, and in substantiating previous discussions of this important subject, the authors have rendered a valuable service. The paper is especially timely because of the great interest, in recent years, in ground water as a source of water supply and of the more general appreciation of the fact that this resource is not inexhaustible. The treatment of the subject relating to ground water occurring under water-table conditions is so effective that it is regretted that the paper was limited in scope so as not to include a discussion of the equally important subject of ground water under artesian conditions.

It is noted that the authors make no reference to the paper by D. G. Thompson and A. G. Fiedler, Assoc. M. Am. Soc. C. E., on "Some Problems Relating to the Legal Control of the Use of Ground Waters," which was awarded the John M. Goodell prize of the American Water Works Association for the most meritorious paper published in its *Journal* during 1938.[60] This paper discusses some of the fundamental principles of ground-water hydrology which should be understood in any attempt to outline a sound doctrine of legal control of the use of ground water. In a discussion of the paper by Harold Conkling, M. Am. Soc. C. E., referred to by the authors, Mr. Thompson[61] has also pointed out, by discussion of cases that have already been decided and of conditions observed in other parts of the country where litigation has not yet arisen, the inconsistencies that result from the unscientific classification of ground waters that have crept into the law.

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[59] Geologist in Charge, Div. of Ground Water, Geological Survey, U. S. Dept. of the Interior, Washington, D. C.

[60] *Journal*, Am. Water Works Association, Vol. 30, 1938, pp. 1049–1091.

The erroneous legal concepts regarding ground waters were first presented to the courts at a time when true scientific concepts had not yet been well developed and doubtless largely by persons who were not familiar with the knowledge of the subject that was then available. These erroneous concepts have been perpetuated because of the reluctance of the courts to depart radically from precedents established in previous cases. The matter of precedent confronted the framers of the New Mexico ground-water law enacted several years ago, which declared ground water to be subject to appropriation in that state. As a result of an intensive investigation of the Roswell artesian basin by the U. S. Geological Survey, in cooperation with the state engineer of New Mexico, the scientific facts regarding the occurrence of the ground waters of that area were thoroughly determined. Nevertheless, recognizing the strong influence that the precedent of earlier decisions has upon the courts, the framers of the law were loath to disregard the more or less generally accepted legal terminology that had been supported by earlier decisions. Therefore, the law was made to apply to "the waters of underground streams, channels, artesian basins, reservoirs, or lakes, having reasonably ascertainable boundaries." It is understood that this terminology was intended to be sufficiently broad to include the so-called "percolating water" under the doctrine of appropriation. Such a broad interpretation would seem to be justified because all bodies of ground water can be included under one or more of the terms "streams, channels, artesian basins, reservoirs, or lakes" and the boundaries of virtually all bodies of ground water can now be determined with reasonable accuracy.

The authors discuss "Effects on Surface Flow of Pumping from Influent and Effluent Subflow"; they do not discuss the converse subject which might be stated "Effects on Subflow of Diversions from Influent and Effluent Surface Streams." In most cases it is the user of ground water who has been on the defensive; and unless the evidence has been clear that there was no interference, the case has generally been decided in favor of the user of the stream water. However, if the ground-water constitutes a part of the stream before it appears as surface water, does not that same ground water also constitute a part of the stream for purposes of appropriation? Accordingly, if the person who pumps subflow is required to desist in favor of a prior appropriator of surface flow, should he not in turn be given priority over a subsequent appropriator of the surface flow? Questions of this kind have arisen in the Mokelumne River area, in California, and in the Platte River Valley, in Nebraska, where heavy pumping from wells was practiced before the diversions from the rivers were made. A careful consideration of all factors involved must lead to the conclusion that eventually, in many areas, the surface water rights—whether by court adjudication or by appropriation—must be evaluated in relation to the rights to use ground water, and vice versa.

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[61] Discussion by D. G. Thompson of "Administrative Control of Underground Water: Physical and Legal Aspects," by Harold Conkling, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 102 (1937), pp. 798–819 (especially pp. 803–811).

M. R. LEWIS,[62] M. AM. SOC. C. E. (by letter).—The difficulties, to which the arbitrary and artificial classification, by the courts, of ground water into "subsurface streams" and "percolating waters" has led, are well presented in this paper. It is to be hoped that as the trend toward state control of the development and use of ground water proceeds, legislatures and courts will be guided by a genuine understanding of ground-water hydrology.

In Part I, under the heading "Effects on Surface Flow of Pumping from Influent and Effluent Subflow," the authors state that, "Regardless of whether influent or effluent conditions existed prior to the development of such a cone of depression below stream bed, the pumping well is supplied by diversion of subflow plus water furnished by the induced influent seepage from the surface stream." This statement should be qualified to the extent of excluding those influent streams where the water table is not in contact with the surface stream. In general, further lowering of a water table that has already lost contact with a surface stream will not increase the stream loss. This point is correctly stated in the paper under "Subsurface Water Course: Influent and Effluent Conditions."

In the case of a subsurface stream whose banks are permeable, but less so than the channel deposits, the writer does not agree with the statement that.

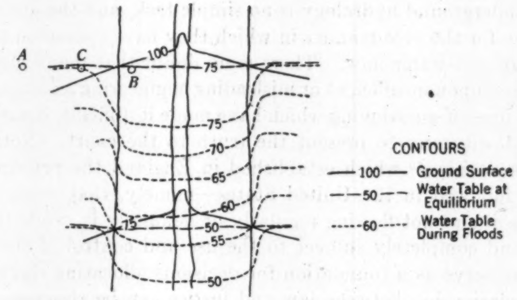

FIG. 11.—WATER-TABLE CONTOUR PATTERN IN A SUBSURFACE CHANNEL HAVING LESS PERVIOUS BANKS

"When flood flow produces a pronounced mound in the subsurface stream, the water-table contours beyond the subsurface banks are also bent into a less pronounced influent shape, indicating slight influent movement from subflow into the older valley fill" (see Part I, heading "Subflow Indicated by Contour Pattern").

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[62] Senior Agri. Engr., SCS, U. S. Dept. of Agriculture and Oregon State College, Corvallis, Ore.

It is believed that the shape of the contour close to the subsurface bank in the less pervious material will be more, rather than less, pronouncedly influent. From Fig. 11 it will be evident that water-table contours plotted from readings in wells *B* and *A* alone would indicate a flatter slope of the water table outside than inside the old channel, whereas readings in *B* and *C* would show the much steeper slope to be expected in the more impervious material. As is inferred in the sentence quoted, the movement of water into the less permeable banks will be less rapid than in the channel deposit, but this will be true because of the decreased permeability and in spite of a steeper water-table gradient.

Similarly in the case of the trench in the subflow channel indicated in Fig. 10, the contours just outside of the subsurface channel should be more nearly parallel to the bank than the contours inside the channel.

To enable the courts to consider the general applications of ground-water hydrology and law when they are called upon to decide specific cases, it would be well if appropriate state or federal agencies were permitted and empowered to appear as "friends of the court" in order to present such general applications, and thus protect the general public from the ill effects that might result from decisions based on incorrect or incomplete presentation of facts or faulty reasoning.

The authors note that no recent text on water-right law is available. It should be noted that Wells A. Hutchins, irrigation economist, Division of Irrigation, Soil Conservation Service, has prepared such a text.[63]

Bayard F. Snow,[64] M. Am. Soc. C. E. (by letter).—Analyzing erroneous concepts of underground hydrology is no simple task, and the authors deserve commendation for the able manner in which they have presented a number of examples of ground-water flow. There is no doubt that many legal decisions have been based upon insufficient or misleading engineering assistance and have in turn led to lines of questioning which have made it difficult, if not impossible, for competent witnesses to present the truth to the court. Notable in this respect is the decision[65] which established in England the principle that has been largely followed in the United States—namely, that water percolating through the soil and not flowing regularly or definitely is, while there, a part of the land and completely subject to the use and control of the owner. If this paper can serve as a foundation for decisions allocating rights to ground waters, the relationship between law and justice can be closer and a forward step can be taken in the conservation of one of the most valuable and least understood of the natural resources.

There is, perhaps, a danger that in the simplified examples given a reader, particularly if looking for an explanation to fit his conditions, may overlook certain fundamental facts. Removal of part of the runoff, whether from a well or from a surface stream, will diminish the total flow from a watershed by the amount so removed. The effect of such removal may be felt, immediately below, entirely in the ground-water body or in the surface stream, or in part by each; but farther downstream the effect is almost certain to be divided between the two.

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[63] "Selected Problems in the Law of Water Rights in the West," by Wells A. Hutchins, U. S. Dept. of Agriculture, *Miscellaneous Publication No. 418* (publication pending).

[64] Cons. Engr., Santurce, Puerto Rico.

[65] Acton vs. Blundell, 12 M. and W. 324.

Under natural conditions, surface streams are maintained and augmented by effluent flow of ground water, generally gaining in volume as they progress toward the ocean; but, at places in their courses where ground water is lower than surface water level, they may lose volume to such extent as the permeability of the bed and banks permits. The relationship between rainfall, evaporation, and transpiration will determine the changes in volume of the total runoff; the cross-sectional area, permeability, and slope of the aquifer will fix its hydraulic capacity and consequently the "water table" for a given volume, subject to the influence of strata of less permeable material. These less permeable strata may result in a perched water table or in an artesian condition.

The authors have called attention to the disagreement between legal and scientific definition of "percolating waters." The definition of "tributary percolating waters" described by Kinney[12, 14] (see heading "'Percolating Waters': Subsurface Reservoir") would deny to mankind the use of "percolating waters," since waters which do not "form a vast mass of water * * * moving slowly down to lower levels," are not available for pumping or diversion as the authors state.

The law has divided available ground water into two classifications and has differentiated between the rights which may be acquired in accordance with that classification; but if the cited definition is to govern, rights to "percolating waters" are of no practical value. The authors say that "'subsurface streams,' 'percolating waters,' and 'artesian waters,' as defined by lawyers, all move by percolation in interconnected openings of capillary size." Exception might be made in the case of cavernous rock channels forming "subsurface streams," but these are not under consideration at the moment. Available ground waters are percolating waters, and the writer's experience has made him very reluctant to answer legal questions about underground streams. Beds and banks of freely permeable aquifers may be relatively impervious or may be only slightly less permeable than the body of the stream. Coarse, water-bearing deposits of gravel and sand frequently disclose no connecting link of permeable material, no matter how extensive the underground exploration may be. Such connecting links between aquifers which appear to be definitely separated, either horizontally or vertically, by almost impervious deposits can be found to exist by pumping tests, and, if confined in cross-sectional area, may be "subsurface streams" of relatively large hydraulic capacity; but response of an observation well to a distant pumping well does not in itself warrant the conclusion that such a stream, with definite "bed and banks," exists. On the other hand, the connection may be large in cross section and without definite boundaries but gradually merging with a less pervious material. In such cases the "bed and banks" do not exist. In either case the entire cross section of the ground-water body is similar, in many respects, to a surface stream in which the velocity decreases markedly as one approaches the banks. If such a stream is partly obstructed by vegetation or porous materials, velocities will be greatly reduced, particularly through the obstructed part of the section. The flowing waters, however, whether in the less obstructed channel or among the weeds and obstructions near the shore, are all part of the surface water body. One can scarcely believe that a court could differentiate between the more freely flowing and the more obstructed waters in allocating rights for diversion.

In the New England area, at least, the coarser glacial deposits are frequently found to be in random lenses or pockets, and the relatively large and permeable potential aquifers are separated, horizontally and vertically, by finer and less permeable materials. If natural conditions produce little differential head, communication between two such deposits will not be induced to any marked degree; but if a considerable head is created by pumping, the flow between the two permeable deposits by what was originally a vein only slightly more pervious than surrounding material will be developed so that it will tend to become an "underground stream," the bed of which may be made relatively impervious by the addition of the finer grains moved from the interstitial spaces in the cross section and redeposited in interstices in the bed. The legal definition of "underground stream," however, takes into account the permanence of the channel, and it appears that states which have distinguished between "percolating waters" and "underground streams," with respect to prescriptive rights, have applied to the latter, when in a "distinct, permanent and well defined channel," the same rules as apply to surface streams. If permanence is essential to the legal existence of an underground stream, and if the disturbance of natural conditions attendant on the development of a well tends to destroy evidence as to whether any such stream did in fact previously exist, or even creates such a stream or concentration of flow, proof of that legal existence may be difficult. Wells alongside of surface streams have drawn water which could conclusively be shown to be surface water although the surface stream had bed and banks of fine, tight, water-resisting material. In other cases, wells similarly located drew only ground water. Except for the physical and chemical characteristics, the origin of the well water could not be determined. Similarly, an underground stream, flowing by a "defined and known channel" (using "defined" in the legal sense of "a contracted and bounded channel, although the course of the stream may be undefined by human knowledge"), may or may not contribute to the yield of a well, and differentiation of rights by the legal classification of underground waters is impracticable.

Although "all underground waters are presumed to be percolating, and to take them out of the rule with respect to such waters the existence and course of a permanent channel must be clearly shown," the distinction is one of degree and is not absolute; and the writer agrees wholeheartedly with the authors that "A few general rules of law founded on well-established hydraulic engineering principles should eliminate much of the confusion that has arisen in the attempt to follow prior legal rulings based on an arbitrary classification not in agreement with occurrence of water below ground surface." The foregoing can be illustrated by reference to court decisions in at least eight states: Florida,[66]

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[66] Tampa Waterworks Co. vs. Cline, 37 Fla. 586, 20 So. 780, 53 Am. St. Rep. 262, 33 L. R. A. 376.

Iowa,[67] Maryland,[68] Mississippi,[69] Ohio,[70] Oregon,[71] West Virginia,[72] and Wisconsin.[73]

The writer recalls litigation involving legality of a taking of a well site under an old charter which granted right to take "* * * the water of any stream or streams, or of any spring or springs * * *." After tests by 2.5-in. driven wells, the taking was made and a gravel-packed well constructed. The plaintiff contended that the well did not take waters of either stream or spring. It was obvious that the well did not take water from a stream or spring, whether one includes underground streams or not, and also whether those words are as understood by the layman or have any special meaning in the profession or trade. However, for two reasons it appears that the water supply was within the limits set by the charter. First, the wording "stream or streams, spring or springs" was evidently intended for what, in later years, would have been described as any sources of surface or ground water. Second, any available ground water is water in motion toward a point of discharge at a lower level, which point of discharge is a spring, so that ground water naturally is the water of a spring or springs.

Another result of legal misunderstanding of ground-water hydrology and of legal precedents that have been set up (and a very serious one) is illustrated in decisions which may be summarized as follows: Connecticut,[74] New York,[75] Michigan,[76] and Kentucky.[77,78]

Wells and underground waters may be contaminated by disposal of gas house wastes, oil, and other substances deposited in or on the ground in such a way that they leach through the earth and pollute the ground-water body. In some jurisdictions no relief from such contamination was granted, inasmuch as intent to injure or negligence could not be proved. One judicial comment indicates a tendency to be guided by public policy rather than legal theory, basis for decision which should appeal to engineers only if the public policy is well founded and recognizes the importance of ground water and the conditions which will affect its quantity or quality and any future needs. In another case cited, the right to destroy a well by withdrawing water from adjoining land is taken as precedent for a right to corrupt the waters, since the injury "is the same kind and degree in the two cases." In all of these cases the decision took into account the legal classifications of ground waters.

It is obvious that although a few decisions have recognized that all underground waters are presumed to be percolating, in general, the courts have distinguished between percolating waters and underground streams. In the normal course, these decisions will be precedent to more and more opinions, all based upon an erroneous concept, a distinction which has existed in the minds of men but not in the forces of nature.

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[67] Barclay vs. Abraham, 121 Iowa 619, 96 N. W. 1080, 100 Am. St. Rep. 365, 64 L. R. A. 255.

[68] Western Maryland R. Co. vs. Martin, 110 Md. 554, 73 Atl. 267.

[69] Clarke County vs. Mississippi Lumber Co., 80 Miss. 535, 31 So. 905.

[70] Wyandot Club vs. Sells, 9 Ohio S. and C. Pl. Dec. 106, 6 Ohio N. P. 64.

[71] Taylor vs. Welch, 6 Oreg. 198.

[72] Pence vs. Carney, 58 W. Va. 296, 52 S. E. 702, 112 Am. St. Rep. 963, 6 L. R. A. N. S. 266.

[73] Huber vs. Merkel, 117 Wis. 355, 94 N. W. 354 (see 48 Central Digest title "Waters and Watercourses," No. 109).

[74] Brown vs. Illius, 27 Conn., 84, 71 Am. Dec. 49.

[75] Dillon vs. Acme Oil Co., 48 Hun. 565, 2 NYS 289, 291.

[76] Upjohn vs. Richland Board of Health, 46 Mich. 542, 9 N. W. 845, 848, 41 Am. Rep. 178.

[77] Kinnaird vs. Standard Oil Co., 89 Ky. 468, 12 S. W. 937, 7 L. R. A. 451, 25 Am. St. Rep. 545.

[78] Long vs. Louisville and Nashville R. Co., 128 Ky. 26, 107 S. W. 203, 13 L. R. A. (N. S.) 1063, 16 Am. Cas. 673.

The fundamental necessity of water to all living beings makes it important that ground-water resources be understood not only by those who use them and their technical advisers, but also by those who fix the rights to their use. It is important that ground-water hydrology be better known in order that rights to such water may properly be allocated among owners on any ground-water body. It is even more important, however, to prevent the loss of that ground-water body to man by its contamination. A misunderstanding which results in unjust allocation of water may affect only two owners, and that only during the period of unjust diversion. Other owners farther downstream may still have an adequate supply for their needs. There may even be adequate supply for the disputing owners at some additional expense. The problem, in short, is whether one or the other shall utilize the available water without interference. In such a problem the public has no basic interest. On the other hand, a misunderstanding of ground-water flow which results in legal permission for the contamination of the ground-water body and of the earth through which it percolates not only affects the adjoining owner but may destroy the value of the ground-water body throughout its entire course, and even for years after the contamination is discontinued, although that body might otherwise have been an adequate and satisfactory source for many users.

Contrary to the foregoing opinion, withdrawal of water from an excavation on adjoining lands is unlikely to destroy a well, at least beyond remedy, except by interception of what would legally be an underground stream or of substantially the entire ground-water body, whereas contamination of the ground water can certainly destroy the neighbor's well beyond remedy. The writer believes that the implications of the decisions permitting contamination of percolating waters are far more serious and inimical to the public good than any allocation of those waters, no matter how erroneous its basis, to what presumably is a purpose useful to mankind.

The authors have called attention to the paper by Mr. Conkling.[3] It would be well for any person interested in the legal phases of ground water to review that excellent paper and its discussion. It may be significant that both the present paper and that by Mr. Conkling came from California and that most of the discussion dealt with western conditions; but although the west has had more reason to be interested in both legal and hydrologic questions pertaining to ground water than have the more humid sections of the country, such questions are of vital importance to many in New England and elsewhere along the Atlantic Coast. Even if in some cases the interests in the East may be superficially in surface waters only, ground water in motion is only water which was surface water and is on its way to an outlet where it will again be surface water; and "all water beneath the surface of the ground, after all, is purely and simply ground-water, moving according to certain well recognized laws of physics. There seems to be no scientific reason why an elaborate and expanding classification of ground-waters should be necessary."[79]

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[79] Discussion by D. G. Thompson of "Administrative Control of Underground Water: Physical and Legal Aspects," by Harold Conkling, *Transactions*, Am. Soc. C. E., Vol. 102 (1937), p. 809.

Ground water and surface water being interdependent, and there being no scientific justification for dividing the former into the legal classifications which have only resulted in confusion, it follows that every one who is directly or indirectly interested in water (and who is not?) should be concerned with the problem of ground-water administration. David G. Thompson's suggestions[30] as to initiating action are most apt. He not only directs attention to the necessity of conducting discussions on a broad basis, without restrictions along lines of location or of use or misuse of ground-water resources, but also lists other associations which include members interested in this condition, referring finally to the legal profession, since "when once the majority of the technologists agree, the position of the lawyer is to show not what the desired result shall be, but how it can be brought about."

Harold Conkling,[31] M. Am. Soc. C. E. (by letter).—This discussion relative to the subject matter of the author's excellent paper is limited mainly to a typical stream, in an alluvial filled valley, flowing underground or partly on the surface in its more normal regimen. It does not concern itself with the phenomena associated with the occasional large flood which may course through a surface channel so rapidly that there is not time for conditions to adjust themselves.

Study of ground water makes revision of the definition of such a stream desirable. A former concept is expressed in the quotation which the author cites (see heading "Subsurface Water Course"): "A 'stream of water' has a defined channel. It has banks, and is very distinct from the percolations of subsurface water, which oozes in veins or filters through the earth's strata."

This definition describes only the surface portion of the stream. The entire stream may be many times wider and deeper and may include "the percolations of subsurface water." It may extend downward and laterally into the alluvium. It may have a bank or banks, but not necessarily so. If so, the bank may be a sudden and definite change from permeable alluvium to material so fine as to be practically impermeable; or the bank may be the bedrock or other material that bounds the valley and rises as hills on either side. The boundary of the stream may be indicated only by a radical change in direction of flow of the underground water, and in such case it is not definite. Neither is a definite boundary readily discernible when two bodies of water flowing on the surface meet unless the waters are of different color. Although it may not be definite, yet if information is sufficient, it will be evident that the boundary is within a zone which can be delimited and this is true whether the water is on the surface or underground.

The water in the surface manifestation of a stream moves at varying speeds. At one point it may be stationary and nearby in rapid movement. Generally it is quietest near its banks. Its particles move vertically and laterally with reference to the general body of the water. Any particle may be above ground here and below ground there, or the reverse. The dominating phenomenon is that the particles move as a whole in the same general direction, although the direction of part of them may be reversed for a short time. A study of the movement of surface streams will show some of the many variations.

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[30] Discussion by D. G. Thompson of "Administrative Control of Underground Water: Physical and Legal Aspects," by Harold Conkling, *Transactions*, Am. Soc. C. E., Vol. 102 (1937), p. 818.

[31] Deputy State Engr., Water Rights, Sacramento, Calif.

The following definition of a stream is offered: "A stream is a body of water moving as a whole in an accustomed location and in a definite direction." This defines the Gulf Stream, or any ocean current, as well as a land stream, and the definition must be thus broadened because a fresh-water stream in alluvium is often (in fact, generally) a body of water moving over and through alluvium filled with water. The portion of the surface manifestation of the stream which flows the fastest is water moving through water and, in spite of the interference caused by the particles of alluvium, the underground portion is water moving through water just as the Gulf Stream is a body of water moving through water.

This conception and definition does not agree with Figs. 9 and 10 and the text in Part I, under the heading "Subflow Indicated by Contour Pattern." As indicated by the water-table contours, the boundary of the stream is not the dashed line shown in the figures but some other more distant line.

The quotation cited by the author to which reference has heretofore been made goes on to state that "An 'underground stream' of water differs from a surface stream only with respect to its location above or below the surface." This is correct if the concept of the surface stream is as the writer has outlined in the foregoing paragraphs. However, since the concept of a surface stream in the mind of the author of the quotation is faulty, his concept that water moving through alluvium beneath the surface is necessarily not a stream is also faulty. With reference to a line at right angles to the axis of direction of movement, a so-called surface stream may be (and generally is) partly underground; an underground stream is wholly underground. Again, with reference to a similar line, the underground stream may be partly a surface stream a few rods either upstream or downstream from the observer or the partly surface stream may be an underground stream.

In the typical underground basin of the mountainous and more or less arid region of the United States, the underground water may have its source in several streams which enter the basin, and each one of these may produce a subsurface stream which coalesces with the others to form a main stream that moves to the basin outlet. In other words they are tributaries. Water is a comparatively nonviscous fluid which moves rapidly with the various forces acting upon it; but underground where it is capable of moving only through the small interstices of the alluvium which impedes it, it acts as a highly viscous fluid, and its rate of movement as compared to the rate of movement of water on the surface may be almost infinitely slow. Therefore, as compared to a so-called surface stream carrying the same amount of water, the underground stream may be very wide and deep and the water table may have a much greater slope than the surface portion of a stream.

The conclusion from the foregoing is this: Generally, conflicts over use of ground water arise only after its use has become extensive, and consequently there probably exists a considerable body of information from wells and surface indications as to its movement. Except for a slight technical difference that it is unnecessary to discuss herein, ground water moves under the influence of gravity as surface water does, except that its rate of movement is much slower and the phenomena of rapid movement are absent. Since the phenomena associated with the movement of surface water are well known, trustworthy deductions can be drawn in many cases as to the movement of underground water even if all information concerning it is necessarily derived from measurements at wells defining the water table at those points.

With the information now available, and its crystallization into the conception of a stream hereinbefore expressed, it would seem probable that, in the future, the subsurface water involved in many cases can be treated as a stream by experts and courts, thus removing the concepts in previous decisions which the author mentions.

As a further matter to consider, it may be that hydrologists, geologists, and engineers have used a different nomenclature too freely in describing underground water and that, with the accompanying and necessary definitions, confusion instead of clarification has resulted. Although it may be acted on by different forces in different locations, water is water whether on or below the earth's surface and the phenomena associated with the movement of surface water are part of the experience of most men.

C. F. TOLMAN[82] AND AMY C. STIPP[83] (by letter).—The writers have received both benefit and pleasure from the discussions of their paper. These discussions have been in two forms: Formal written contributions published in *Proceedings*, and informal discussion by personal letter or conversation.

The personal discussions were more critical than those published, and showed widely divergent viewpoints. For example, an engineer criticized the paper because of its limited scope, and opinion was expressed that the value of the contribution was greatly reduced by the lack of a discussion of confined water. On the other hand, a lawyer thought the paper covered far too large a field, and suggested that the writers limit discussion to the "support" of stream flow by ground water.

The writers appreciate that only one small phase of the application of ground-water hydrology to legal concepts was treated, and that the hydrology of confined water and analysis of the decisions regarding it are of increasing importance to the engineer and lawyer.

In both the personal discussions and published discussions, the presentation of the effects of pumping on influent and effluent streams was criticized as incomplete. The writers attempted to show that pumping influent subflow not in contact with the stream would not affect surface stream flow as far as influent conditions extend; and that a contraction of stream channel farther downstream would force to the surface a subflow diminished by pumping and thus indicate in the diminished surface flow that water had been removed from subflow.

The writers did not mention the case of a pump situated at a narrow part of the stream channel, with enlargement of stream gravels above and below the narrows. No increased seepage loss due to pumping is registered by measurement of influent seepage below the narrows. Above the narrows, pumping might change effluent conditions, produced by channel contraction, to influent conditions, and measurements of stream seepage loss above the narrows and the pumping plant would register a diminution of surface flow above the narrows.

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[83] Tucson, Ariz.

The writers approve most of the points made in the discussions and, furthermore, many valuable features beyond the scope of the paper were introduced. Only some of the important contributions, and a few of the points on which there is disagreement, will be mentioned.

Mr. Baker mentions that difficulties in California procedure due to faulty legal concepts of ground-water hydrology still exist because many cases were tried and many decisions rendered during the dry cycle at the turn of the century, and that the advances in ground-water hydrology from 1920 to 1940 have not as yet modified the erroneous concepts on which the earlier decisions were founded.

He presents an excellent mathematical method of classifying ground-water movement according to the degrees of confinement of ground-water flow. He attributes many of the recent cases in Southern California to opposition to the lowering of ground-water level by those claiming a superior right to ground water to those possessing inferior rights. He rightfully emphasizes that this demand that the water table must not be lowered may be as absurd as prohibiting the lowering of water level in a surface reservoir during a dry period.

Mr. Wiel's contribution is undoubtedly the most valuable discussion of legal phases of the application of ground-water hydrology. The writers appreciate the exhaustive research that he has made on the subject of ground-water law, and are especially pleased that he has presented so clearly the difficulties in apportioning water according to priority of appropriation, because the doctrine of rights by appropriation has received support by many engineers and lawyers.

Mr. Wiel shows great skill in comparing statements of two authorities on the same subject, attempting to prove, thereby, disagreement in general principles. As a matter of fact, the apparent disagreement usually lies in the difficulty of discussing a complex subject in precise language, with a careful definition of conditions under which the statement holds true. For example, the writers' statement that ground water never occurs as a stationary water body is compared with Mr. Meinzer's suggestion[30] that "some of the water may be virtually stationary in synclinal troughs or encased lenses." This suggestion of Mr. Meinzer is of very limited application, whereas the writers' statement applies to all bodies of ground water in pervious alluvial material.

Mr. Wiel quotes Messrs. Baker and Conkling's statement that the proper conception of ground water in alluvium is that of water in a reservoir, whereas the writers emphasize objections to this concept. Mr. Wiel quotes the senior writer's statement[32] that no water table can exist in impervious material, whereas Mr. Meinzer indicates that in most cases no material is absolutely impervious. "Impervious" and "relatively impervious" have been used more loosely than almost any other hydrologic terms.

Mr. Harris' approval of the scientific foundation, not only for legal decisions but in the preparation of cases, is especially valuable. In the next to the last paragraph he states:

> "The trend in the law following the development of large areas of arid lands through irrigation, with its accompanying creation of new and changed water tables, requires thorough and accurate knowledge of ground water, not only in settling disputes as to the rights to use of such ground water, but in its control for the beneficial and most economical use, and in the preservation of lower lying lands from destruction by too high a water table. For this courts and water users must look to irrigation engineers. The fountainhead for their learning is the universities. Are the universities fulfilling this requirement in the education of their engineers who desire to specialize in irrigation? Are these engineers being qualified as experts in ground water? It is the understanding of the writer that, generally, they are not."

The writers believe that unfortunately engineering instruction in most colleges does not treat, adequately, the laws and principles governing the occurrence and motions of underground water.

Mr. Treadwell criticizes three cases cited for discussion by the writers. An introductory paragraph to this criticism is as follows:

> "Practically every case cited is attacked as being in some way in violation of the positive rules of hydrology developed by the paper. This probably indicates that the scientist has as much difficulty in understanding a judicial decision as the lawyer has in stating a scientific principle. Such a paper would be of much more value if it had been prepared in cooperation with a lawyer. The writer disagrees entirely with the criticisms made of the decisions in question."

The writers assure Mr. Treadwell that they made every attempt to secure the cooperation of an attorney versed in ground-water law, but were not entirely successful.

Mr. Treadwell takes exception to the writers' criticism of the decision in Maricopa County Municipal Water Conservation District No. 1 vs. Southwest Cotton Company,[20] which may have been somewhat biased and harsh. The senior writer testified in the trial of this case, and, in spite of attempts to maintain the independent position of a scientist, his personal experience may have been reflected in his comments on the decision. However, the discussion and principal criticism were directed to the legal "test" or "rule" set forth by the judge to determine the existence of a subsurface stream. Even a vivid geological imagination cannot conceive of so perfect an equilibrium that pumping from subflow abstracts exactly the same quantity of water from the surface flow.

The discussion of two cases, Vineland Irrigation District vs. Azusa Irrigation Company,[21] and Lemm vs. Rutherford,[22] shows that the writers were quite unsuccessful in explaining the behavior between influent and effluent streams and the water tables adjacent thereto. The latter case involved the right to construct a sump in which seepage from a nearby ditch was collected, and from which the water was taken. As seepage was taking place the ditch was influent. As stated in the paper, if the mound developed by seepage water is not in contact with the bottom of the ditch, lowering the ground-water level by drainage will have no effect on the ditch. If a ground-water mound developed by seepage is in contact, and the water-table slope away from the ditch is steepened, then there will be increased seepage. Mr. Treadwell states:

> "Of course, all earthen ditches seep and the amount of the seepage is due to the porosity of the material through which they pass. Even a porous material is some support for the water in the ditch; otherwise all the water would seep from the ditch. The water that fills the pores of the material also acts as a support. If that support is taken away by a trench or sump near the ditch from which trench or sump water is pumped, then there would seem to be room for the claim that such artificial works induced seepage and thus took away the right of natural support. The writer can see no reason why this could not occur."

This is an excellent example of apparently good reasoning that overlooks the two fundamental hydraulic factors: (1) Whether or not the ground-water mound is in contact with the bottom of the stream; and, if so, (2) whether or not the slope of the mound is increased by the construction of the sump.

Mr. Treadwell states that

> "* * * he would like to read further evidence to support the positive rule announced by the authors that as long as there is a column of unsaturated material between the surface stream and the underground water table, a lowering of the water table will not affect the surface stream."

The writers would state that the existence of a water table below the bottom of an influent stream has been proved by measurements of depth to water table in test holes sunk in a stream bottom within the area covered by water. It has been found that measurements of stream loss (influent seepage) do not vary with distance between stream bottom and water table.

Mr. Treadwell suggests "* * * that the water table is at all times supplying this column with water, by capillarity, for several feet above the water table." A close study of Fig. 3 shows a subsurface "rainfall" generated at stream bottom and supplying the water table. The seepage column is a zone of descending ground water and is considered to be in the unsaturated zone because it is above the water table. However, probably most of the pore space is occupied by the influent seepage. Neither the water table nor capillarity furnishes water to this influent seepage column.

As is to be expected, Mr. Meinzer's discussion contains so many pertinent remarks with which the writers are in accord that there is no need to mention them here. He notes that the writers discuss effects on surface flow of pumping from influent and effluent subflow, but do not discuss in detail effects on subflow of diversions from influent and effluent surface streams. Although the principles governing these two cases are brought out in the paper, Mr. Meinzer's discussion of the importance of the effect of stream diversions on ground water corrects one of the defects in the presentation, caused by a desire for brevity.

Discussing the statement in the paper regarding development of cone of depression below stream bed, Mr. Lewis states: "This statement should be qualified to the extent of excluding those influent streams where the water table is not in contact with the surface stream."

It is interesting to note that, although the writers emphasized the importance of contact of ground water with stream bottom, they omitted the statement of this controlling feature in this one case, and they thank Mr. Lewis for bringing it to their attention. He also draws the contours of a ground-water mound due to stream flood, showing that the bend of contours takes place largely in the adjacent, less pervious material, rather than in the channel.

Mr. Snow's criticism has been read with particular interest because it presents the viewpoint of one versed in ground-water conditions in Eastern United States. The general ground-water equation, which evaluates rainfall, runoff, evaporation, and transpiration, can be applied better in the East than in the desert region where these factors are more variable. The study of ground-water supplies in pervious stream gravels incased in less pervious material is based chiefly on stream-flow measurements that give quantity of influent and effluent seepage.

Mr. Snow's interesting discussion of ground-water conditions in the East, and legal problems related thereto, are too numerous to admit of detailed discussion. His statement that the problem of pollution of ground water is generally more important than quantity of ground water applies to conditions in the humid East rather than in the semiarid West.

The discussion by Mr. Conkling is of value because his concept of a ground-water stream is much broader than that of the writers. He offers the following definition: "A stream is a body of water moving as a whole in an accustomed location and in a definite direction." Such a broad view would include deep artesian flow as a ground-water stream or any concentrated flow of ground water not connected or related to a surface stream.

This is a strictly engineering concept, and it would be necessary only to measure the rate and direction of ground-water movement to prove a subsurface stream. The writers' conceptions are geologic, and are based on the occurrence of ground water beneath the valleys and productive bottom lands of the major streams of Western United States.

This geologic concept is as follows: Many of the major streams of the West have broad alluvial valleys with marginal terraced lands several miles in width. These lands are underlain by pervious gravels deposited in a river-cut channel that has been excavated either in bedrock or in less pervious alluvial materials. Under these conditions, ground water moving in this pervious material may be properly classified as a subsurface stream which can be separated from the general body of percolating ground water beyond the outermost banks of the original channel, first excavated and later filled by the river.

It is only to occurrences such as this that the concept of a subsurface stream can be applied, and even in such cases difficulties may arise in separating the ground-water stream from the surrounding body of percolating water.

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TRANSACTIONS

Paper No. 2117

PRESSURE-MOMENTUM THEORY APPLIED TO THE BROAD-CRESTED WEIR

BY H. A. DOERINGSFELD,[1] ESQ., AND C. L. BARKER,[2] ASSOC. M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. J. C. STEVENS, H. G. WILM, I. M. NELIDOV, JOHN W. HACKNEY, THOMAS H. PRENTICE, BORIS A. BAKHMETEFF, D. D. CURTIS, CARL ROHWER, JOHN HEDBERG, AND H. A. DOERINGSFELD AND C. L. BARKER.

SYNOPSIS

The general theory of pressure momentum, and its application to the broad-crested weir, is presented in this paper, with data obtained by test. The purpose of the experimental work was to check a formula for flow over the weir developed on the basis of conservation of momentum. The application also applies to the sharp-edged entrance to flumes from reservoirs.

INTRODUCTION

The advantages of the broad-crested weir as a measuring device, where head is at a premium, have been enumerated by several experimenters and most recently by James G. Woodburn,[3] M. Am. Soc. C. E. The pressure-momentum theory offers a method of developing a formula for the flow of water which is free from empirical coefficients that vary widely with different heads and crest heights. The use of the formulas requires only the measurement of the head on the weir.

Notation.—The symbols used in this paper are defined where they first appear and are assembled for reference in the Appendix.

REVIEW OF THE PRESSURE-MOMENTUM THEORY

Consider the part of a filament of water, $\Delta l'$ in length, whose cross-sectional areas at points 1 and 2 are ΔA_1 and ΔA_2, respectively (see Fig. 1). Let the pressures at points 1 and 2 be P_1 and P_2, respectively. If the inclination of

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[3] "Tests of Broad-Crested Weirs," by James G. Woodburn, *Transactions*, Am. Soc. C. E., Vol. 96 1932), p. 387.

the filament to the horizontal is θ, the resultant force in the direction of motion (that is, downward and to the right) will be the algebraic sum of the gravity and pressure components in that direction. Equating the total force to the product of mass and acceleration (which is the rate of change of momentum):

$$P_1\,\Delta A_1 - P_2\,\Delta A_2 + \Delta G \sin\theta = \Delta m \frac{dV}{dt} \qquad (1)$$

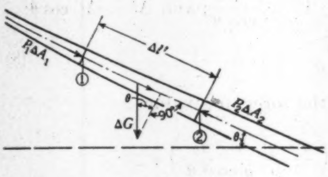

FIG. 1

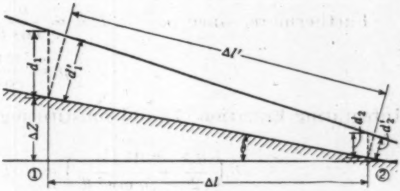

FIG. 2

in which m is the mass; G is the gravatational force; V is the velocity; and t is time. Since the volume of the segment is nearly $\frac{(\Delta A_1 + \Delta A_2)\,\Delta l'}{2}$,

$$\Delta G \sin\theta = \tfrac{1}{2}\,(\Delta A_1 + \Delta A_2)\,\Delta l'\,w \sin\theta \qquad (2)$$

in which w is the weight of water per unit volume. Substituting Equation (2) in Equation (1), multiplying by dt, and integrating:

$$[P_1\,\Delta A_1 - P_2\,\Delta A_2 + \tfrac{1}{2}(\Delta A_1 + \Delta A_2)\,\Delta l' \sin\theta\,]\,t = \Delta m\,V_2' - \Delta m\,V_1' \qquad (3)$$

in which V_1' and V_2' are the velocities at points 1 and 2, respectively, in the direction $\Delta l'$. Substituting $\Delta m = \frac{\Delta Q\,w\,t}{g}$, dividing by t, and transposing:

$$\left(P_1\,\Delta A_1 + \frac{\Delta Q\,w\,V_1'}{g}\right) - \left(P_2\,\Delta A_2 + \frac{\Delta Q\,w\,V_2'}{g}\right) + \tfrac{1}{2}\,(\Delta A_1 + \Delta A_2)\,\Delta l'\,w \sin\theta = 0 \qquad (4)$$

in which g = acceleration due to gravity.

Consider now an open channel of length Δl, depth d, width b, and inclination θ (Fig. 2). From Equation (4), assuming parallel filaments of flow,

$$\sum_0^{d_1'}\left(P_1\,\Delta A_1' + \frac{\Delta Q\,w\,V_1'}{g}\right) - \sum_0^{d_2'}\left(P_2\,\Delta A_2' + \frac{\Delta Q\,w\,V_2'}{g}\right) + \sum_0^{0.5(d_1'+d_2')} \frac{(\Delta A_1' + \Delta A_2')}{2}\,\Delta l'\,w \sin\theta = 0 \qquad (5)$$

but

$$dA_1' = b\,\mathrm{d}d_1' \qquad (6a)$$

$$dA_2' = b\,\mathrm{d}d_2' \qquad (6b)$$

$$dQ = b\,\mathrm{d}d_1'\,V_1' = b\,\mathrm{d}d_2'\,V_2' \qquad (6c)$$

in which b = the breadth of the channel. In Equation (5), replacing the Δ's by differentials and substituting the values in Equations (6):

$$\int_0^{d_1} b\left[P_1\, dd_1' + \frac{dd_1'\,(V_1')^2}{g}\,w\right] - \int_0^{d_2'} b\left[P_2\, dd_2' + \frac{dd_2'\,(V_2')^2}{g}\,w\right] + \frac{b\,(d_1' + d_2')}{2}\,\Delta l'\,w \sin\theta = 0 \qquad (7)$$

Furthermore, since $p = w\,d$; $d = \frac{d'}{\cos\theta}$; $V' = \frac{V}{\cos\theta}$; and $\Delta l = \Delta l' \cos\theta$:

$$p = \frac{w\,d'}{\cos\theta} \qquad (8)$$

Integrating Equation (7) and substituting the foregoing values:

$$b\left(\frac{d_1^2}{2} + \frac{V_1^2\,d_1}{g\cos^2\theta}\right)w - b\left(\frac{d_2^2}{2} + \frac{V_2^2\,d_2}{g\cos^2\theta}\right)w + b\,\frac{(d_1 + d_2)}{2}\,\Delta l\,w \sin\theta = 0 \qquad (9)$$

If $\theta = 0$ and $\frac{V^2}{g} = 2\,H$, the pressure-momentum equation for a horizontal channel becomes:

$$b\,w\left[\left(\frac{d_1^2}{2} + 2\,H_1\,d_1\right) - \left(\frac{d_2^2}{2} + 2\,H_2\,d_2\right)\right] = 0 \qquad (10)$$

in which H = the hydraulic head. If friction forces are considered, the right-hand member of Equation (10) is not zero, but is set equal to the sum of these forces, f. Hence, if

$$F = b\,w\left(\frac{d^2}{2} + 2\,dH\right) \qquad (11)$$

then

$$F_1 - F_2 - f = 0 \qquad (12)$$

in which F is a pressure-plus-momentum function.

APPLICATION OF THEORY TO BROAD-CRESTED WEIR

Consider the broad-crested weir sketched in Fig. 3. The pressure-plus-momentum function in Equation (8), on the upstream side, is:

$$F_1 = b\,w\left(\frac{d_1^2}{2} + \frac{Q^2}{g\,b^2\,d_1}\right) \qquad (13a)$$

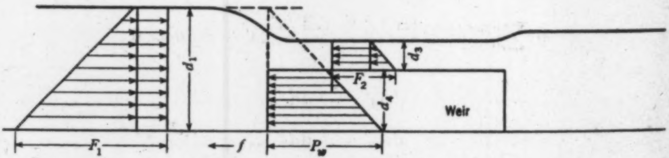

FIG. 3

On the top of the weir:

$$F_2 = b\,w\left(\frac{d_3^2}{2} + \frac{Q^2}{g\,b^2\,d_3}\right) \ldots\ldots\ldots\ldots\ldots\ldots (13b)$$

The force P_w in Fig. 3, due to unit water pressure p on the weir face, is assumed to be:

$$P_w = w\,b\,d_4\left[\frac{d_1 + (d_1 - d_4)}{2}\right] = w\,b\,d_4\left(\frac{2\,d_1 - d_4}{2}\right) \ldots\ldots (14)$$

The accuracy of this assumption is checked experimentally. Substituting Equations (13) and (14) in Equation (12), and simplifying:

$$\left(\frac{d_1^2}{2} + \frac{Q^2}{g\,b^2\,d_1}\right) - \left(\frac{d_3^2}{2} + \frac{Q^2}{g\,d_3\,b^2}\right) - \left(\frac{2\,d_1 - d_4}{2}\right) d_4 = 0 \ldots (15)$$

In Equation (15), let $d_1 - d_4 = H$; and $H = K\,d_3$, in which K = a constant. Then simplifying and solving for Q:

$$Q = \frac{b\,\sqrt{2\,g}}{2}\,H^{1.5}\left\{\left(\frac{K^2 - 1}{K^2}\right)\left[\frac{d_4 + H}{K\,(d_4 + H) - H}\right]\right\}^{0.5} \ldots\ldots (16)$$

In order to reduce Equation (16) to the form commonly developed for weirs, it may be written:

$$Q = \Omega\,b\,H^{1.5} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

in which

$$\Omega = \frac{\sqrt{2\,g}}{2}\left\{\left(\frac{K^2 - 1}{K^2}\right)\left[\frac{d_4 + H}{K\,(d_4 + H) - H}\right]\right\}^{0.5} \ldots\ldots\ldots (18)$$

Assuming the flow across the weir to be parallel, the pressure distribution on the upstream face to be as assumed, and the frictional losses between the crest and the position of parallel flow to be negligible, Equation (16) should give the discharge. To check these assumptions and to evaluate K the following apparatus was developed.

DESCRIPTION OF THE APPARATUS AND TEST PROCEDURE

Minnesota Tests.—A metal weir was constructed of two standard 6-in. channels and one ¼-in. steel plate 1.68 ft by 2.25 ft. The channels that formed the upstream and downstream faces were screwed to the plate that formed the top of the weir. The resulting crest height of the completed weir was 0.53 ft; the length of the weir was the width of the channel, 1.68 ft; and the breadth was 2.25 ft. Holes for piezometer connections were drilled on the center lines of both ends and the top. On the faces, the top hole was drilled as near the edge as possible; the first five holes were spaced 0.025 ft on centers and the remaining holes, 0.05 ft on centers. On the crest, holes were drilled as near the edge as possible; the first five holes from the edge were spaced 0.05 ft on centers and the remaining holes were spaced 0.1 ft on centers. Into each hole drilled in the weir was screwed a brass nipple carrying a hole $\frac{1}{16}$ in. in diameter. The ends of the nipples terminating on the surface of the weir were made very smooth so that there would be no disturbance due to them.

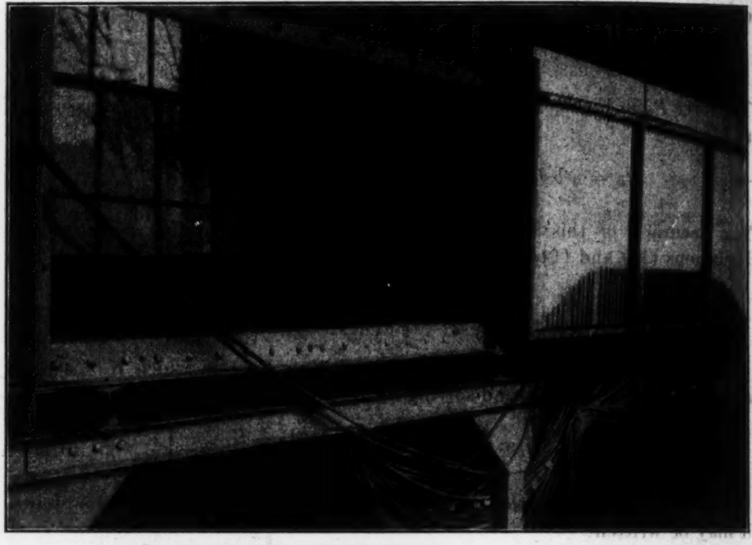

Fig. 4.—The Metal Broad-Crested Weir and Piezometer Tubes

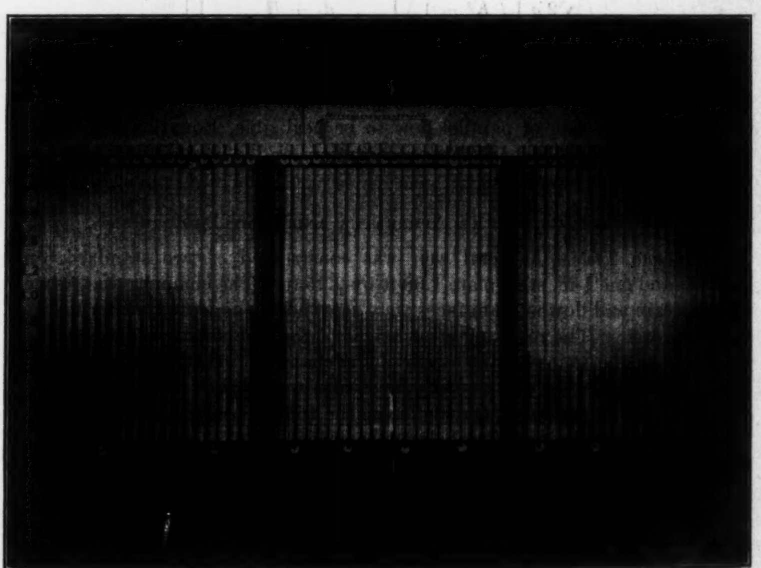

Fig. 5.—The Bank of Piezometer Tubes in Operation Showing the Pressure on the Upstream Face and Top of the Weir

The rubber tubes led from the weir nipples to a rack containing sixty glass piezometer tubes ½ in. in diameter (see Figs. 4 and 5). Two of the glass tubes were connected to read the head on the weir 15 ft upstream. Two more were connected to read the head several feet downstream from the weir.

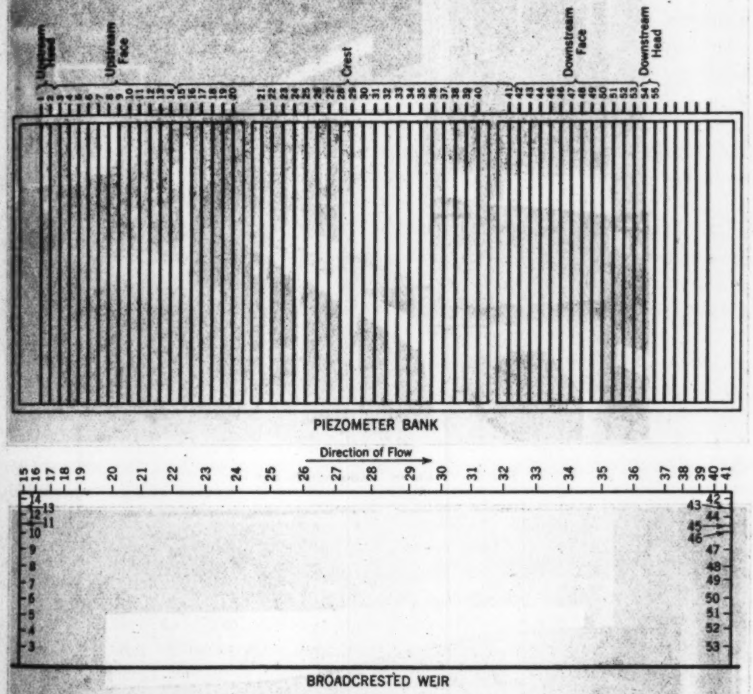
Upstream Head
Upstream Face
Crest
Downstream Face
Downstream Head
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20
21 22 23 24 25 26 27 28 29 30 31 32 33 34 35 36 37 38 39 40
41 42 43 44 45 46 47 48 49 50 51 52 53 54 55
PIEZOMETER BANK
Direction of Flow
15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34 35 36 37 38 39 40 41
14 13 12 11 10 9 8 7 6 5 4 3
42 43 44 45 46 47 48 49 50 51 52 53
BROADCRESTED WEIR

FIG. 6.—BROAD-CRESTED WEIR SHOWING NIPPLES AND NUMBERS; NOS. 1 AND 2 CONNECTED UPSTREAM AND NOS. 52 AND 53 CONNECTED DOWNSTREAM

Fig. 6 shows the location of the nipples on the weir and the numbers of the nipples. These nipples were connected by the rubber tubes mentioned to the correspondingly numbered piezometer tubes in the rock. Piezometer tubes 1 and 2 were connected upstream to measure the head. Nipples 3 to 14 inclusive were on the upstream side.

A scale reading to 0.01 ft was placed behind the piezometer tubes. With the apparatus thus constructed it was possible to read the pressure profile over the weir at a glance. The water-surface profile was taken with a point gage mounted on a track on top of the glass channel (see Figs. 4 and 7). In Fig. 7 the sharp-crested weir is in the foreground. This leads to the glass channel and on to the broad-crested weir. The rails carrying the profile gage are shown on top of the channel.

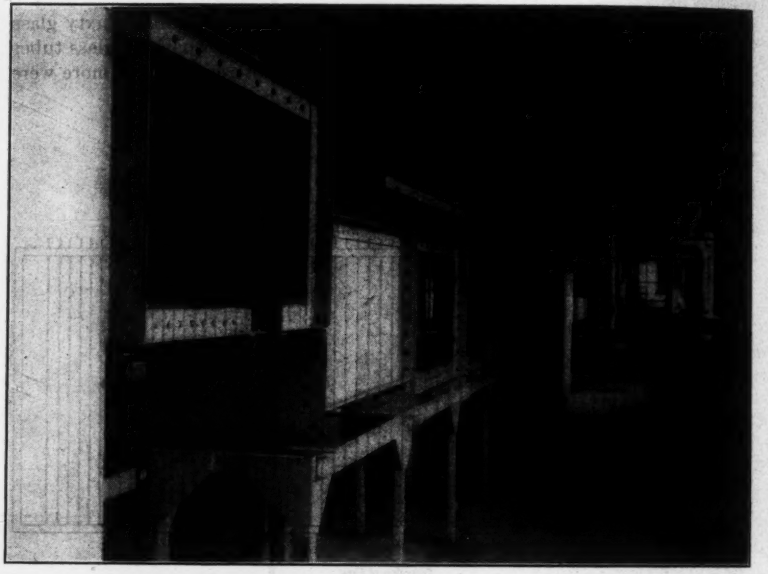

FIG. 7.—VIEW OF LABORATORY SET-UP

0
5
10
15
20
25

FIG. 8.—DIRECTION OF THE FLOW LINES AS INDICATED BY THE THREADS IN NEEDLES

To trace out the flow lines, 2-in. strands of No. 50 black silk thread tied in the eyes of No. 9 needles, which were mounted perpendicular to a white board in columns 1 in. apart, were used. The needles were spaced vertically $\frac{1}{4}$ to $\frac{1}{2}$ in. Photographs were taken of the threads trailing in the stream to provide a permanent record (see Fig. 8).

The water used in the experiments was circulated by pumps in the basement of the laboratory. These forced the water to an overhead constant-level tank out of which the water was carried through a 10-in. header to a sharp-crested rectangular weir (see Fig. 7). The head on the weir was measured by a hook gage. The discharge from the weir was carried by a glass channel 2.5 ft deep, 1.68 ft wide, and 40 ft long. The broad-crested weir under test was placed just downstream from the middle of the channel. The lower end of the glass channel was closed by a gate that could be adjusted to regulate the tailwater elevation on the broad-crested weir. The channel discharged into two 11,000-lb weighing tanks. Before any experimental work was performed, the tanks were calibrated and then used to calibrate the sharp-crested weir at the entrance of the channel. This calibration was checked frequently during the experiment to obviate any errors.

Eleven runs were made under heads varying from about 0.1 ft to 0.6 ft, for each of which the water-surface profile was taken with the point gage. On these runs pressures were read on the weir by means of the piezometers described.

Five weirs of different crest heights, and the same crest length and breadth as the steel weir just described, were placed in the glass channel. The data are shown graphically in Fig. 9.

Washington Tests.—Two weirs were constructed of tempered hard board, with crest heights of 0.262 and 0.471 ft, crest lengths of 0.833 and 0.833 ft, and crest breadths of 1.12 and 1.65 ft, respectively. These weirs were placed in a channel so arranged that the water-surface profile could be determined. The water flowing through the channel was measured by a sharp-crested, constricted weir whose head was measured to 0.001 ft. The head on the broad-crested weir was decreased in nearly uniform increments until standing waves formed on the surface of the weir.

RESULTS

Fig. 10 shows the results obtained as to water depth and pressure distribution over the weir. They check, substantially, the results observed by Professor Woodburn with regard to the limiting depth of tailwater without affecting the head on the weir; and the position of critical depth (d_c) agrees quite well with the locations obtained in his experiments. However, the critical depth was computed on the basis of parallel flow by the formula

$$d_c = \frac{\sqrt[3]{q^2}}{g} \qquad \text{(19)}$$

which cannot be assumed to be the critical depth for curvilinear flow, the condition that exists at the upstream edge of the weir.[4] In Equation (19) q = flow through a unit of cross section of area.

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[4] "Hydraulics of Open Channels," by Boris A. Bakhmeteff, M. Am. Soc. C. E., p. 43.

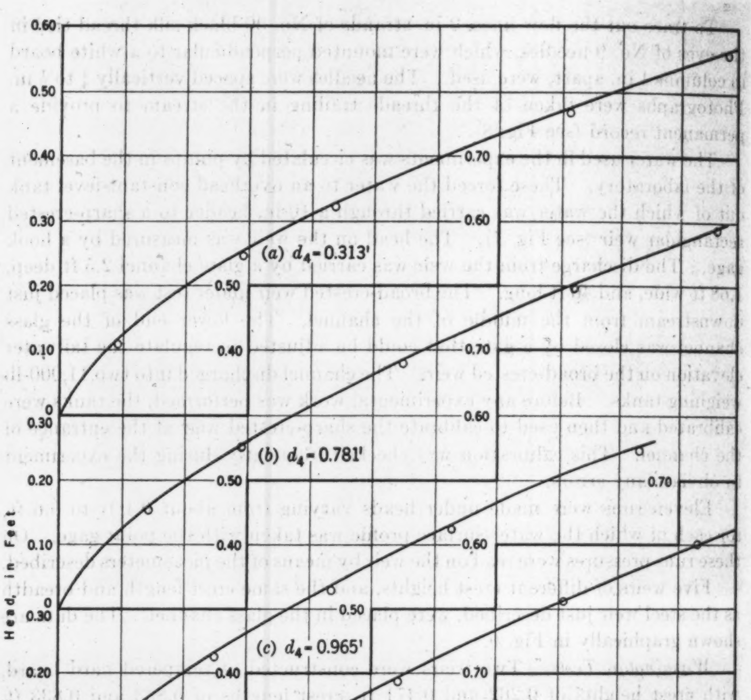

FIG. 9.—CURVES COMPUTED FROM EQUATION (16) AND THE CORRESPONDING EXPERIMENTAL DATA

An examination of the pressure profiles substantiates the assumption made in the development of Equation (15) that the pressure on the upstream face was due to the depth d_1. The study of the water-surface profiles and the lines of flow taken from the photographs gave the positions of d_3 (depth of parallel flow across the weir) for the different runs. At the point of minimum depth, for a very short stretch, the water is flowing parallel (or nearly so) to the surface of the weir. It was at this point that d_3 was measured.

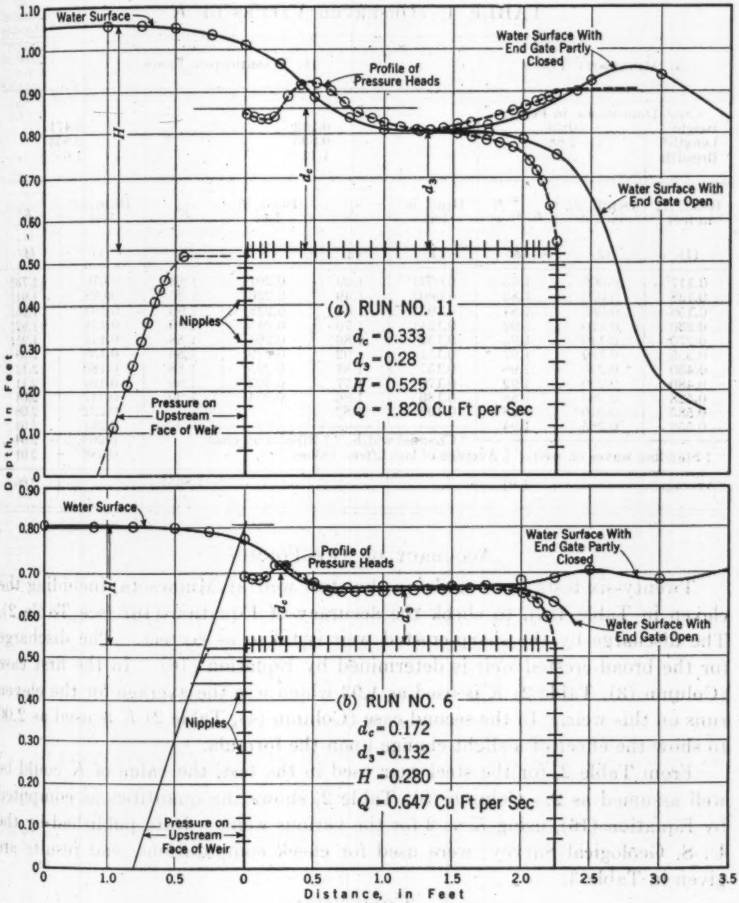

FIG. 10.—WATER DEPTHS AND PRESSURE DISTRIBUTION

The values of K on the Minnesota weir are given in Table 1(a), and Table 1(b) shows the values of K determined at Washington State College. Although there is some variation in the average values of K determined, the formula is rather insensitive to the value of K. The average of all the various values where there were no standing waves on the weir is 1.93. If a value of 2 is used for K, to make computations simple, the results given by Equation (16) are quite close for practical computations.

TABLE 1.—OBSERVED VALUES OF K

(a) MINNESOTA TESTS			(b) WASHINGTON TESTS					
Crest Dimensions, in Feet:								
Height	0.53		0.262				0.471	
Length*	1.68		0.833				0.833	
Breadth	2.25		1.12				1.65	
Head, H, in feet (1)	Depth, d_3, in feet (2)	$K = \frac{H}{d_3}$ (3)	Head, in feet (1)	K (3)	Head, in feet (1)	K (3)	Head, in feet (1)	K (3)
0.117	0.060	1.95	0.053†	1.29	0.207	1.83	0.076	1.73‡
0.128	0.070	1.83	0.090‡	1.49	0.226	1.86	0.088	1.91‡
0.156	0.085	1.84	0.103‡	1.64	0.228	1.90	0.108	1.96‡
0.230	0.120	1.92	0.124‡	1.70	0.245	1.90	0.121	1.92‡
0.270	0.140	1.93	0.132	1.86	0.268	1.88	0.144	1.95‡
0.355	0.180	1.97	0.151	1.92	0.279	1.86	0.159	2.09
0.430	0.220	1.96	0.157	1.84	0.295	1.88	0.180	2.12
0.480	0.250	1.92	0.170	1.77	0.302	1.90	0.194	2.11
0.528	0.280	1.88	0.186	1.86	0.334	1.90	0.217	2.04
0.585	0.310	1.88	0.195	1.82	....	....	0.232	2.08
0.552	0.285	1.94					0.251	2.04
							0.268	2.04
							0.288	2.01
Average	....	1.91	....	....	....	1.865§	....	2.06

* Channel width. † Ripples on crest.

‡ Standing waves on weir. § Average of last fifteen values.

ACCURACY OF THE FORMULA

Twenty-six tests were made on the steel weir at Minnesota, including the eleven in Table 1(a), to check the accuracy of Equation (16) (see Table 2). The discharge by the sharp-crested weir is taken as correct. The discharge for the broad-crested weir is determined by Equation (16). In the first case (Column (3), Table 2) K is used as 1.93 which was the average for the eleven runs on this weir. In the second case (Column (4), Table 2) K is used as 2.00 to show the effect of a slight change upon the formula.

From Table 2, for the steel weir used in the test, the value of K could be well assumed as 2. Column (9), Table 2, shows the quantities as computed by Equation (16), using K as 2 for the various weirs. Data published by the U. S. Geological Survey[5] were used for check computations, and results are given in Table 3.

CONCLUSION

It is the writers' opinion, based on the tests and computations given, that flow over a broad-crested weir may be computed quite accurately by use of Equations (16) and (17), assuming K to be 2. Thus far tests were made on only sharp-edged weirs. Undoubtedly a similar expression could be developed for round-edged weirs. The accuracy of the formulas is to be questioned if the breadth of the weir crest is so small that the water surface does not become

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[5] *Water Supply Paper No. 200*, U. S. Geological Survey, p. 70.

TABLE 2.—TESTS TO CHECK THE ACCURACY OF EQUATION (16)

Head on broad-crested weir, in feet; crest height d_4 = 0.53	DISCHARGE BY SHARP-CRESTED WEIR, IN CUBIC FEET PER SECOND			PERCENTAGE ERROR, BY EQUATION (16)		Head on broad-crested weir, in feet; crest height d_4 = 0.262	DISCHARGE BY SHARP-CRESTED WEIR, IN CUBIC FEET PER SECOND			PERCENTAGE ERROR, BY EQUATION (16)	
	Observed	By Equation (16)		K = 1.93	K = 2.00		Observed	By Equation (16) (b = 0.833; and d_4 = 0.262)*		K = 2.00	K = 1.865‡
		K = 1.93	K = 2.00					K = 2.00	K = 1.865‡		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
0.068	0.075	0.0760	0.0750	+1.30	0.00	0.053*	0.0270	0.0262	0.0264	− 2.86	−2.22
0.112	0.160	0.1612	0.1617	+0.60	+1.06	0.090*	0.0650	0.0592	0.0599	− 9.50	−7.90
0.114	0.165	0.1670	0.1660	+1.20	+0.73	0.103*	0.0790	0.0732	0.0740	−13.00	−6.33
0.117	0.170	0.1745	0.1732	+2.60	+1.76	0.124*	0.1000	0.0977	0.0993	− 2.30	−0.70
0.128	0.193	0.2000	0.1983	+3.50	+2.62	0.132	0.1100	0.1078	0.1093	− 2.00	−0.64
0.144	0.240	0.2410	0.2380	+0.40	−0.83	0.151	0.1348	0.1331	0.1355	− 1.27	+0.52
0.155	0.270	0.2690	0.2667	−0.37	−0.11	0.157	0.1420	0.1415	0.1435	− 0.35	+1.05
0.156	0.270	0.2720	0.2693	+0.74	−0.26	0.170	0.1650	0.1602	0.1626	− 2.90	−1.56
0.178	0.330	0.3340	0.3300	+1.20	0.00	0.186	0.1810	0.1844	0.1886	+ 1.88	+4.20
0.230	0.500	0.4950	0.4940	−1.00	−1.30	0.195	0.2040	0.1990	0.2021	− 2.45	−0.93
0.270	0.647	0.6410	0.6340	−0.90	−2.00	0.207	0.2270	0.2189	0.2222	− 3.96	−2.12
0.289	0.700	0.7120	0.7080	+1.70	+0.87	0.226	0.2620	0.2513	0.2557	− 4.08	−2.40
0.334	0.890	0.8940	0.8860	+0.45	−0.45	0.228	0.2615	0.2550	0.2605	− 2.49	−0.39
0.355	0.955	0.9830	0.9730	+3.00	+2.30	0.245	0.2910	0.2853	0.2907	− 1.95	−0.10
0.390	1.140	1.1470	1.1300	+0.60	−0.88	0.268	0.3350	0.3290	0.3354	− 1.79	+0.12
0.430	1.280	1.3330	1.3180	+4.10	−3.14	0.279	0.3500	0.3509	0.3575	+ 0.26	+0.21
0.434	1.290	1.3500	1.3380	+4.70	+3.72	0.295	0.3850	0.3830	0.3908	− 0.52	+1.49
0.452	1.420	1.4430	1.4250	+1.60	+0.35	0.302	0.4180	0.3972	0.4053	− 5.02	−3.11
0.480	1.560	1.5580	1.5670	−1.80	+0.45	0.334	0.4550	0.4669	0.4765	+ 2.53	+4.73
0.481	1.580	1.5900	1.5720	+0.60	−0.51	0.108*	0.0770	0.0768†	0.0752†	− 0.26	−2.24
0.508	1.730	1.7400	1.7200	0.60	0.58	0.121*	0.0940	0.0915†	0.0900†	− 2.66	−4.25
0.528	1.820	1.8500	1.8230	+1.60	+1.65	0.144*	0.1200	0.1197†	0.1176†	− 0.25	−2.00
0.552	1.973	1.9800	1.9550	+0.35	−0.91	0.159	0.1440	0.1395†	0.1373†	− 3.12	−4.65
0.555	1.975	1.9950	2.0770	+1.01	+0.52	0.180	0.1680	0.1692†	0.1663†	+ 0.72	−1.01
0.587	2.130	2.1800	2.1570	+2.35	+1.27	0.194	0.1820	0.1898†	0.1871†	+ 4.10	+2.80
0.588	2.150	2.1900	2.1620	+1.40	+0.56	0.217	0.2300	0.2260†	0.2225†	− 1.74	−3.26
						0.235	0.2530	0.2560†	0.2521†	+ 1.19	−0.36
						0.251	0.2830	0.2837†	0.2798†	+ 0.25	−1.13
						0.268	0.3020	0.3140†	0.3097†	+ 4.00	+2.55
						0.288	0.3470	0.3515†	0.3468†	+ 1.30	−0.05

* Ripples and standing waves on crest.
† In the last eleven tests d_4 = 0.471.
‡ In the last eleven tests K = 2.08.

TABLE 3.—EXPERIMENTS ON WEIRS OF IRREGULAR SECTION

(a) BAZIN'S SERIES NO. 115: CREST HEIGHT = 2.46 FT; AND CREST LENGTH = 6.56 FT						(b) CORNELL EXPERIMENTS:* CREST HEIGHT = 11.25 FT; AND CREST LENGTH = 15.969 FT		
Observed head, in feet	Coefficient $C_1 = \frac{Q}{L\,H^{1.5}}$	Value of Ω by Equation (18)	Observed head, in feet	Coefficient $C_1 = \frac{Q}{L\,H^{1.5}}$	Value of Ω by Equation (18)	Observed head, in feet	Coefficient $C_1 = \frac{Q}{L\,H^{1.5}}$	Value of Ω by Equation (18)
(1)	(2)	(3)	(1)	(2)	(3)	(1)	(2)	(3)
0.196	2.25	2.50	1.023	2.61	2.65	0.312	2.684	2.47
0.264	2.41	2.52	1.097	2.63	2.67	0.447	2.713	2.48
0.342	2.45	2.53	1.178	2.64	2.68	0.698	2.696	2.49
0.415	2.51	2.55	1.260	2.65	2.69	0.851	2.579	2.50
0.495	2.50	2.57	1.330	2.68	2.70	1.206	2.652	2.52
0.566	2.55	2.58	1.388	2.69	2.71	1.480	2.531	2.53
0.638	2.54	2.59	1.424	2.70	2.71	1.890	2.624	2.54
0.716	2.56	2.61	1.467	2.70	2.72	2.360	2.637	2.57
0.792	2.60	2.62				2.935	2.608	2.60
0.871	2.60	2.63				3.661	2.637	2.62
0.948	2.60	2.64				4.436	2.647	2.65

* *Water Supply Paper No. 200*, U. S. Geological Survey, Series XLIII, p. 70.

parallel to the surface of the weir, or nearly so. Raising the surface of the tailwater had no effect on any runs until the point of parallel flow was shifted from the position it had with the tailwater down. Hence, the formulas can be used where some of the head is being regained by raising the tailwater.

ACKNOWLEDGMENT

The experiments upon which this paper is based were originally started at the Experimental Engineering Laboratory, University of Minnesota, Minneapolis, Minn., under the direction of Lorenz G. Straub, Assoc. M. Am. Soc. C. E. Later experiments were made in the Hydraulic Laboratory at the State College of Washington, Pullman, Wash. Other sources of information are cited by footnote references in the text.

APPENDIX

The following notation conforms essentially with American Standard Symbols for Hydraulics,[6] compiled by a committee of the American Standards Association with Society representation, and approved by the Association in 1929.

A = area of cross section; ΔA = increment of area;
b = breadth;
C = coefficient; $C_1 = \frac{Q}{L\,H^{1.5}}$;
d = depth of flow; d' = depth perpendicular to the bottom slope; d_c = critical depth; and d_1, d_2, and d_3 are as shown in Fig. 3;
F = pressure-plus-momentum force;
f = sum of all friction forces;
G = gravitational force; ΔG = increment force;
g = acceleration due to gravity;
H = hydraulic head;
K = a constant $= \frac{H}{d}$;
L = length;
l = length of a filament of water; Δl = increment of length;
m = mass;
P = total pressure; P_1 and P_2 = total hydraulic pressure on the cross sections at points 1 and 2, respectively; P_w = force due to water pressure on the weir face;
p = unit pressure intensity;
Q = rate of flow;
q = rate of flow through a unit cross section;
t = time;
V = velocity; V' = velocity of a given point, in the direction of $\Delta l'$;
w = weight of water per unit volume;
θ = inclination of a water filament with the horizontal; also, θ = slope of the bottom of the channel.

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[6] A. S. A.—Z10b—1929.

DISCUSSION

J. C. STEVENS,[7] M. AM. SOC. C. E. (by letter).—The results of some interesting and practical experiments are reported in this paper. The authors have presented a rational expression for the coefficient C in the well-known weir formula, $Q = C\,L\,H^{1.5}$. Their mathematical approach to this expression, however, merits further consideration. Equation (15) is orthodox and could have been written directly from an inspection of Fig. 3, provided the hypothesis is accepted that the pressure on the upstream face of the weir is that due to the depth d_1. Although this assumption appears to be confirmed by the experiments, it can be considered only an empirical approximation.

There is a downward acceleration of the water at the upstream edge of the weir and, as a falling body loses weight in proportion to its downward acceleration, to that extent is the pressure on the front face reduced below the static pressure due to the depth d_1. In other words the flow at this section is not parallel but curvilinear. This fact is well illustrated by the profile of pressure heads in Fig. 10, which is substantially below the water-surface profile in the section of the upstream edge of the weir. There is also a dynamic pressure on the upstream face of the weir due to the velocity of approach. If this velocity is uniform and equal to V_1, the dynamic pressure is $\frac{w\,b\,d_4\,V_1}{g}(1 - \cos\alpha)$ where α is the angle through which the initial direction of the velocity has been turned by the obstruction. Although the front face of the weir is at right angles to the initial direction of V_1, the value of α is not 90° but something less. There is a horizontal, revolving eddy or cushion in front of the weir which deflects the water upward and over the crest, giving it an upward acceleration as it passes over the edge. This tends to balance the downward acceleration of the surface water. The combined effect is seen in the curved line of pressures against the front face of the weir in run No. 11 of Fig. 10.

Equations (13a) and (13b) are expressions of the pressure-plus-momentum functions—not forces. This function was used effectively by Julian Hinds,[8] M. Am. Soc. C. E., in his analysis of the hydraulic jump. It is also designated as the M-function by Boris A. Bakhmeteff,[9] M. Am. Soc. C. E. The authors have considered the momentum of the water as $\frac{w\,Q\,V}{g}$, whereas if the mean velocity is used a coefficient should be applied. The value of this coefficient depends upon the velocity distribution within the prism of flowing water. It is given by[10]

$$C_m = \frac{\int_0^A v^2\,dA}{V^2\,A} \qquad \text{(20)}$$

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[7] Cons. Hydr. Engr. (Stevens & Koon), Portland, Ore.

[8] "The Hydraulic Jump and Critical Depth in Design," by Julian Hinds, *Engineering News-Record*, Vol. 85, 1920, p. 338.

[9] "Hydraulics of Open Channels," by Boris A. Bakhmeteff, McGraw-Hill Book Co., Inc., New York, N. Y., 1st Ed., p. 234.

[10] "Applied Fluid Mechanics," by M. P. O'Brien, M. Am. Soc. C. E., and George H. Hickox, Assoc. M. Am. Soc. C. E., McGraw-Hill Book Co., Inc., New York, N. Y., 1st Ed., p. 271.

Equation (9) and its preceding development appear to have but slight relation to the remainder of the paper, which is fortunate, for it contains several errors, some of which are doubtless typographical.

The pressure intensity at any point on a sloping bed supporting a stream is $p = w\, d' \cos \theta$, in which d' is the depth normal to the bed, or $p = w\, d \cos^2 \theta$ in which d is the vertical depth. The total pressure force in a unit width for the two cases then becomes

$$P' = w \frac{(d')^2}{2} \cos \theta \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21a)$$

and

$$P = w \frac{d^2}{2} \cos^2 \theta \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21b)$$

Writing the momentum equation in terms of depths normal to the channel bed and velocities parallel thereto, as given in Fig. 2, the expression for a unit width is

$$\left[\frac{(d_1')^2}{2} \cos \theta + C_{m1} \frac{q\, v_1'}{g} \right] + \frac{d_1' + d_2'}{2} \Delta l' \sin \theta$$
$$- \left[\frac{(d_2')^2}{2} \cos \theta + C_{m2} \frac{q\, v_2'}{g} \right] = 0 \ldots\ldots\ldots\ldots\ldots (22)$$

in which C_m is the coefficient of momentum, and q is the flow per unit width.

This may be expressed in terms of vertical depths and horizontal velocities by substituting: $d' = d \cos \theta$; $\Delta l' = \dfrac{\Delta l}{\cos \theta}$; $v' = \dfrac{v}{\cos \theta}$; and $q = v'\, d' = v\, d$. After dividing by $\cos^2 \theta$:

$$\left(\frac{d_1^2}{2} + C_{m1} \frac{v_1^2\, d_1}{g \cos^3 \theta} \right) + \frac{d_1 + d_2}{2} \Delta l \frac{\tan \theta}{\cos \theta} - \left(\frac{d_2^2}{2} + C_{m2} \frac{v_2^2\, d_2}{g \cos^3 \theta} \right) = 0 \ldots (23)$$

which is seen to differ materially from Equation (9). However, for $\theta = 0$, it reduces to the first two terms of Equation (15), since $v^2\, d = \dfrac{Q^2}{b^2\, d}$.

This expression neglects the friction which is an upstream force. If d_2 is less than d_1, there is a downward acceleration; hence the unit weight is not w, but $w \left(1 - \dfrac{a}{g} \right)$, in which a is the downward acceleration. However, as long as the upstream friction force is neglected, the specific weight cancels out as does the width and Equation (23) is correct for any sloping rectangular channel.

The error of omitting $\cos \theta$ in $p = w\, d \cos \theta$ amounts to 1% for an 8° slope and 14% for 30°. Therefore, it must not be omitted in steep chutes and on the face of overflow dams. Wherever the slope is steep enough to take into account the force of gravity as expressed by the last term of Equation (9), then the effect of this slope on the static pressure in the vertical should also be considered as included in Equation (23).

As a metering device, the broad-crested weir may have advantages under some circumstances. The rounding of the upstream curve will probably have the effect of increasing the discharge. The deposit of sand, silt, or sludge against the upstream face will also increase the discharge, since it lessens the upstream reaction to the dynamic force against the front face. It is not applicable without further tests where the tailwater is likely to submerge the crest more than $\frac{2}{3}$ H.

The Parshall flume[11] has decided advantages over this type of weir because it keeps itself generally free of sediment and the calibrations cover sizes from 3-in. to 50-ft crest widths and include both free-flow and submerged-flow conditions. For the former, as in this case, only a single head is observed up to a two-thirds submergence. Beyond that limit, of course, both headwater and tailwater levels must be observed, for which the flows are given in empirical formulas and tables.

There is need for an open-channel metering device which is indexed by far smaller heads than is possible with any type of free-flow weir. The writer believes that the submerged Parshall flume and the submerged orifice[12] are the best devices so far developed to meet such conditions, largely because of the exhaustive calibrations they have received.

H. G. WILM,[13] ESQ. (by letter).—Many attempts have been made to develop devices for water measurement which could be "rated" on a purely theoretical basis, without the need for empirical coefficients to adjust theory to actuality. Until now, as far as the writer knows, no such attempt has been entirely successful because of errors in the basic assumptions involved. One assumption which, perhaps, results in the most deviations of theory from practice, is that water is without viscosity and hence without friction. Another is that acceleration, deceleration, and changes in direction of water flow are accomplished without expenditure of either time or space.

Fig. 3 and the authors' design for the broad-crested weir (Fig. 10) used in their experiment illustrate partial failure to recognize empirical requirements. Throughout the paper, apparently, the assumption is made that parallel flow will occur over the level weir surface, except where the flow is influenced by end conditions. This condition is impossible to obtain in practice. For any given discharge the depth at any point along the weir will depend on the friction of the weir material and on the distance of the point in question from the ends of the weir.

The authors' use of a weir with a sharp upstream corner was due, apparently, to the second of the foregoing assumptions. Since water does require time and space in which to accomplish a sudden change in direction of flow, the inevitable result is flow separation just above and downstream from the sharp corner. The threads in Fig. 8 illustrate this well-known phenomenon. Its significance to their study is simply that discharge over the weir is necessarily influenced by such separation. The actual depth influencing discharge over the weir is not exactly "d_3" in Fig. 3, but an indefinite depth, measured above the bottom of the *vena contracta* rather than above the weir surface.

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[11] *Bulletin No. 423*, Colorado Experiment Station (U. S. Dept. of Agriculture), Ft. Collins, Colo.; see also "The Improved Venturi Flume," by Ralph L. Parshall, Assoc. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 841.

[12] "The Calco Metergate," The R. Hardesty Manufacturing Co., Denver, Colo.

[13] Silviculturist, Rocky Mountain Forest and Range Experiment Station, Fort Collins, Colo.

As a combined result of these two factors, the "minimum depth" (d_1) for any given discharge is necessarily an uncertain quantity, depending for its exact magnitude and location upon velocity of approach, rate of discharge, surface friction, and weir length. Although the resulting errors in measurement of discharge might not be excessive, they should easily be as great (up to 8% or more, depending on the value of "K") as those observed by Messrs. Doeringsfeld and Barker. Since other flow-measuring devices of greater accuracy are available, and since field installations ordinarily show greater variability than laboratory studies, errors of such size would invalidate the authors' method for most purposes.

Professor Woodburn,[14] in the publication cited by the authors, discusses in detail the locations of critical depths on the weirs studied, as follows:

> "The foregoing discussion of the position of critical depth pertains to the weirs in which the entrance was rounded. With a square-cornered entrance the conformation of the water surface over the weir was quite different, as is seen by comparing the profiles of Series *A* with those of Series *B* to *G*. The initial drop in the water surface was greater with the square-cornered entrance than with the rounded entrance preceding a level crest 10 ft. broad. With the square-cornered entrance the water surface dropped to a point below the critical depth and then commenced to rise gradually. This rise continued either until it approached the critical depth, when the surface became wavy and turbulent, or until it reached the vertical curve caused by the fall. The critical depth was reached and waves were formed only with flows of less than about 4 cu. ft. per sec. The waves formed with the rounded entrance were smooth in profile, and the water surface was practically level in cross-section at all points. With the square-cornered entrance the waves reached more of a peak, being considerably higher in the center of the stream than at the sides; and there was more surface turbulence."

Professor Bakhmeteff,[15] in his discussion of Professor Woodburn's paper, effectively clarifies the relation of critical depths and associated phenomena to flow over broad-crested weirs.

Fig. 10 in the paper illustrates, in two ways, the influence of the weir employed upon flow characteristics. First, plotted pressures show violent variation directly above the sharp entrance corner. Wherever curved flow exists, of course, pressures will deviate from depths. The variation shown in Fig. 10, however, is doubtless caused by sharply curved flow at the bottom as well as at the top of the flow column, with the column of actual flow moving an appreciable distance above the weir surface and with a body of relatively dead water between. Second, the "minimum depth" (d_1) is well below both computed and actual critical depths in both illustrated cases. Since the weir surface is level longitudinally, this condition is a purely local one induced by residual momentum and pronounced flow curvature in the entrance zone. If the weir had been longer, flow depths (as described by Professor Woodburn) would have increased downstream from the zone of curvature until they reached characteristic rippled flow at the critical depths. As concluded by the writer and his collaborators,[16] however, flow on a level broad-crested weir is sufficiently unstable to be unsatisfactory for purposes of discharge measurements.

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[14] "Tests of Broad-Crested Weirs," by James G. Woodburn, *Transactions*, Am. Soc. C. E., Vol. 96 (1932), pp. 401–404.

[15] Discussion by Boris A. Bakhmeteff of "Tests of Broad-Crested Weirs," by James G. Woodburn, *Transactions*, Am. Soc. C. E., Vol. 96 (1932), pp. 423–434.

These arguments are by no means intended to indicate that the broad-crested weir is unsuited to measurement of open-channel discharge. This type of structure is excellently adapted to the purpose and should measure discharge with accuracy equal to any other type of device suited to field use. Furthermore, when properly designed and constructed, broad-crested weirs can be calibrated accurately on a rational basis. In order to do so, however, several requirements must be met in design:

(1) All changes in flow direction upstream of the measuring point must be accomplished by well-rounded entrance transitions. The necessary minimum radius of curvature varies with expected velocities of approach.

(2) The point of pressure or depth measurement must lie in a zone of relatively straight-line flow, where pressures and depths show close agreement. In ordinary design, this means that the weir length must be at least three times the greatest water depth to be measured—preferably four to six times—and the point of measurement should be near the longitudinal midpoint of the weir.

(3) The weir surface should have a longitudinal slope sufficiently greater than the critical slope for the weir material so that at all stages rapid, stable flow velocities will occur at the point of measurement. Critical slopes for any weir material may be computed by the following formula:

$$S_c = \frac{g}{C^2} \times \frac{P}{b} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (24)$$

in which S_c is the critical slope; g is acceleration due to gravity; P is the wetted perimeter; b is the top width; and C is the Chézy coefficient corresponding to a given Kutter's friction factor and depth of flow.

(4) The Kutter's friction factor must be known within close limits for the weir material employed. This can be determined either by a simple hydraulic-laboratory test or by obtaining a few measured rating points for a field installation.

With satisfaction of these requirements the broad-crested weir affords a simple and easily calibrated device for measuring flow, and one which is relatively free from influence by velocity and stage conditions either upstream or downstream of the structure. If the weir surface is laid on a longitudinal slope approximately equal to the critical slope for the range of depths employed, discharge should conform with reasonable closeness to the theoretical formula for flow at critical depths in a rectangular open channel:

$$Q = \sqrt{g}\, b\, h_c^{1.5} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (25)$$

The weir must be long enough, however, to permit establishment of stable straight-line flow, with the point of measurement within the zone of stable flow and downstream from the zone of transition influence. Since such flow may exhibit some instability, expressed in surface waves, it is safer to give the weir a slope greater than critical. In this case, a calibration formula may be derived by the method described by Fred C. Scobey,[17] M. Am. Soc. C. E., and outlined by R. L. Stoker.[18] One modification only is suggested, based on experimental observations—that the location of critical depths be assumed to lie along a line, not vertical, but inclined downstream at an angle of approximately 10° from the vertical. The origin of the line should be fixed at the upstream edge of the broad-crested weir (not including any rounded entrance).

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[16] "Measurement of Débris-Laden Stream Flow with Critical-Depth Flumes," by H. G. Wilm, John S. Cotton, and H. C. Storey, *Transactions*, Am. Soc. C. E., Vol. 103 (1938), pp. 1247–1249.

When surface curves for several discharges have been computed by this method, depths may be taken at any desired point of measurement and a calibration formula computed by fitting a logarithmic straight line to the stage-discharge relation. This formula will comply with the form

$$Q = C\, b\, h^{1.5} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (26)$$

in which C is an empirical coefficient to correct for friction losses and for progressive downstream displacement of the critical depth with increasing discharges.

The writer, with others, worked from 1934 to 1938 on the development of a flume for use in measuring open-channel flows containing bed loads. This flume functions on the same principle as a broad-crested weir. The essential difference in design involves only the use of horizontal transitions instead of a bottom contraction to accomplish the development of critical flow. Pressure measurements are taken within the flume, at a point downstream from the zone of critical depths. Since the floor slope is greater than the critical, stable scouring velocities are maintained at the measuring section.

Preliminary results of these studies were published in 1938.[19] Further developments by means of half-scale model studies, as they pertain to the present discussion, may be summarized as follows:

(1) Variations in wall and floor friction from a Kutter's factor (n) of 0.010 to one of 0.015 cause an inverse variation in discharge up to 5%.

(2) Discharges appear to be little affected by variations in approach and exit conditions. Tailwater is tolerated without appreciable effect on discharge, up to 70% to 80% of the initial specific energy of the discharge. (Initial specific energy equals depth plus velocity head, considering the channel bed at flume entrance as zero head.)

(3) For flume lengths shorter than a minimum of twice the distance from entrance transition to piezometer (about six times the greatest measured water depth), the depth-discharge relation changes rapidly. Increase in length of the downstream section, up to a maximum overall length of three times the minimum, appears to have no measurable effect on discharge.

(4) Comparisons were made of the relative accuracy of several means of transmitting pressures to piezometer wells: Static tubes, standard piezometer orifices, and vertically elongated slots in the flume wall. Compared to static-

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[17] "The Flow of Water in Flumes," by Fred C. Scobey, *Technical Bulletin No. 393*, U. S. Dept. of Agriculture, 1933.

[18] Discussion by R. L. Stoker of "Measurement of Débris-Laden Stream Flow with Critical-Depth Flumes," by H. G. Wilm, John S. Cotton, and H. C. Storey, *Transactions*, Am. Soc. C. E., Vol. 103 (1938), pp. 1268–1270.

[19] "Measurement of Débris-Laden Stream Flow with Critical-Depth Flumes," by H. G. Wilm, John S. Cotton, and H. C. Storey, *Transactions*, Am. Soc. C. E., Vol. 103 (1938), p. 1237.

tube data as a check, the slots tested seem to be as reliable as pressure orifices. Variation in width of slot (in the models) from $\frac{1}{32}$ to $\frac{1}{8}$ in., and in wall thickness from $\frac{1}{8}$ to $\frac{1}{2}$ in., appeared to cause no variation in measured pressures. Within the roughness range tested ($n = 0.010$ to 0.015), flume-wall roughness has little or no effect upon orifice or slot readings.

All of the aforementioned studies were conducted during the period 1934–1938, while the writer was a staff member of the California Forest and Range Experiment Station, under its auspices and in cooperation with the University of California, at Berkeley, Calif. They were designed to test and calibrate a particular flume design—that for the San Dimas Flume in its final form as illustrated by Mr. Stoker.[20] The principles involved and the results of these studies, however, should be applicable to any broad-crested weir that conforms in design and construction to the foregoing requirements.

As an illustration, Table 4 presents observed depth and discharge data from model studies of the San Dimas Flume, with computed depths obtained by the aforementioned specific-energy method.

TABLE 4.—COMPARISON OF OBSERVED DEPTHS WITH DEPTHS COMPUTED BY SPECIFIC-ENERGY METHOD; SIX-INCH MODEL OF SAN DIMAS FLUME

(Computations Were Made by Slide-Rule)

Observation No.	Floor slope, S, in feet per foot	Kutter's friction factor, n	Observed discharge, in cubic feet per sec	DEPTHS, IN FEET			Percentage deviation
				Observed	Computed	Deviation	
106	0.031	0.010	0.179	0.116	0.115	−0.001	−0.9
112	0.031	0.010	0.418	0.223	0.223	0.000	0.0
114	0.031	0.010	0.630	0.301	0.303	+0.002	+0.7
115	0.031	0.010	0.923	0.397	0.399	+0.002	+0.5
125	0.050	0.010	0.180	0.107	0.107	0.000	0.0
129	0.050	0.010	0.420	0.205	0.207	+0.002	+1.0
130	0.050	0.010	0.633	0.281	0.284	+0.003	+1.1
131	0.050	0.010	0.917	0.374	0.378	+0.004	+1.1
166	0.050	0.0134	0.180	0.112	0.112	0.000	0.0
168	0.050	0.0134	0.424	0.215	0.220	+0.005	+2.3
169	0.050	0.0134	0.640	0.293	0.298	+0.005	+1.7
191	0.050	0.0152	0.181	0.120	0.121	+0.001	+0.8
189	0.050	0.0152	0.420	0.219	0.222	+0.003	+1.4
187	0.050	0.0152	0.637	0.298	0.299	+0.001	+0.3
188	0.050	0.0152	0.926	0.393	0.402	+0.009	+2.3
				Average percentage deviation = ±0.94.			

The flume involved was the 6-in. model described by Mr. Stoker. Observed depths were obtained by point gage to the nearest 0.001 ft; discharges were measured by means of 4 by 2 in. and 6 by 4 in. venturi meters. For observations Nos. 106, 112, 114, 115, 125, 129, 130, and 131 the flume was of smooth, shellacked and varnished wood, with a friction factor (n) estimated at 0.010. For the other observations given in Table 4, the same flume was roughened with two different sizes of sand. In these cases the value of n was computed by the formula

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[20] Discussion by R. L. Stoker of "Measurement of Débris-Laden Stream Flow with Critical-Depth Flumes," by H. G. Wilm, John S. Cotton, and H. C. Storey, *Transactions*, Am. Soc. C. E., Vol. 103 (1938), p. 1269.

$$n = 0.043\, D^{1/6} \qquad (27)$$

in which D = mean diameter of sand particle, in feet. The floor slope of the flume was 0.031 for the first four observations, and 0.050 for the others.

It will be noted that most of the deviations of computed from observed depths are positive in direction. This should be due either to a slight error in the formula for computing n, or to the incorrect location of critical depths in plotting water-surface curves. However, since flow acceleration is relatively slow within these flumes, considerable longitudinal displacement of the line of critical depths may occur without causing appreciable error in measured depths at the piezometer.

I. M. Nelidov,[21] M. Am. Soc. C. E. (by letter).—The principle of impulse-momentum has been applied by the authors to the flow over a broad-crested weir, and their theoretical results are represented in the formula common to weirs:

$$Q = C\, L\, H^{1.5} \qquad (28)$$

Experiments were also conducted by the authors, and their results, together with the results of experiments made by others, were applied to interpret the theoretical derivations.

The principal conclusion advanced in the paper is that the ratio of depth to head on the weir is $K' = \frac{1}{K} = 0.5$, which permits one to compute the coefficient of discharge C. The latter coefficient varied from 2.25 to 2.70 for Bazin's experiments and from 2.68 to 2.65 in Cornell experiments. The authors did not give their values of the coefficient C. These, computed by the writer from Table 1, vary from 2.50 to 2.87.

The writer wishes to discuss the connection between the specific case chosen by the author and the application of the momentum principle to more general cases of flow over the broad-crested weir of any cross section, with any shape of approach channel, the latter varying from the cross section of the weir to that of the reservoir. Consider a flow, as shown in Fig. 11, over the broad-crested weir at section 3–3 from a channel at section 1–1.

The forces resulting from water pressure and acting on the flow are shown shaded. In the profile in Fig. 11(a) they represent the hydrostatic pressures on sections 1–1 and 3–3 and the reactions of the invert of the transition. In the plan (Fig. 11(b)) they represent the projections of the hydrostatic pressures on sections 1–1 and 3–3, and of the reactions of the sides of the transition.

The hydrostatic pressure P_1 acts in section 1–1, based on depth H above the crest of the weir and its width b_3. At section 3–3 the hydrostatic pressure P_3 acts, in the opposite direction, based on depth d_3 on the weir and its width b_3. Below the level of the weir, and on its sides, the additional hydrostatic pressure in section 1–1 is counteracted by the horizontal projections of the reactions of the invert represented in profile and in plan by dotted lines. The remainder of these pressures, directed downstream, is represented by the force P_0. The force P_0 is a function of d_4, $H - d_3$, and $b_1 - b_3$. Because of the curvature of the water surface between sections 1–1 and 3–3, neither the reactions of the invert nor the force P_0 is composed of linear unit pressures. On the contrary, these pressures vary curvilinearly with depth. This factor reduces the pressure P_0 considerably.

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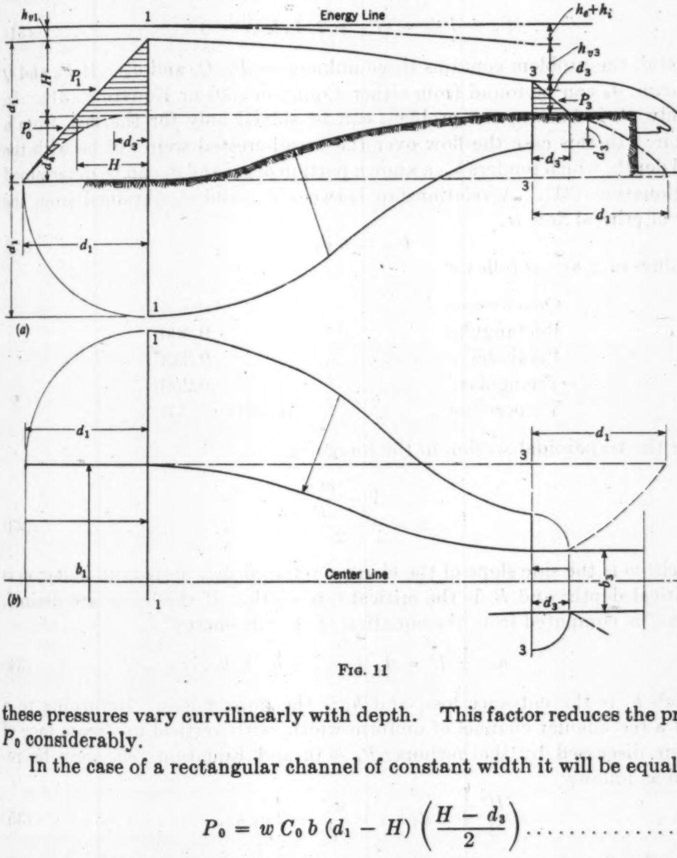

FIG. 11

In the case of a rectangular channel of constant width it will be equal to:

$$P_0 = w\, C_0\, b\, (d_1 - H) \left(\frac{H - d_3}{2} \right) \ldots\ldots\ldots\ldots\ldots\ldots (29)$$

in which C_0 is a coefficient depending on the shape of the invert, and is smaller than unity. The impulse momentum equation for a general case may be written as follows:

$$(P_0 + P_1 + 2\, A_1\, h_{v1}) - (P_3 + 2\, A_3\, h_{v3}) = 0 \ldots\ldots\ldots\ldots (30)$$

in which P_0 is the force due to excess of hydrostatic pressures in the downstream direction, acting below the level of the weir, and on its sides; P_1 is the force due to hydrostatic pressures in section 1–1 above the level of the weir, based on width of weir b_3; P_3 is the force due to hydrostatic pressures in section 3–3; A_1 is the cross-sectional area at section 1–1 based on depth d_1; A_3 is the cross-sectional area at section 3–3, based on depth d_3; h_{v1} is the velocity head at section 1–1; and h_{v3} is the velocity head at section 3–3.

For a case of flow from a reservoir, $h_{v1} = 0$, and Equation (29) becomes:

$$(P_0 + P_1) - (P_3 + 2\,A_3\,h_{v3}) = 0 \tag{31}$$

In general, the problem contains three unknowns: P_0, Q, and d_3. If P_0 and Q are known, d_3 can be found from either Equation (30) or Equation (31). In case only P_0 is known, the problem can be solved only for the flow from a reservoir. In this case the flow over the broad-crested weir will be with the critical depth, which renders h_{v3} a known portion of d_3; and d_3 can be determined from Equation (31). A relationship between h_{v3} and d_3 obtained from the theory of critical flow is:

$$h_{v3} = n\,d_3 \tag{32}$$

The values of n are as follows:

Channel section	Value of n
Rectangular	0.500
Parabolic	0.333
Triangular	0.250
Trapezoidal	Equation (33)

For the trapezoidal section in the foregoing,

$$n = \frac{1 - \dfrac{m\,d_c}{B_c}}{2} \tag{33}$$

in which: m is the side slope of the channel referred to a horizontal unity; d_c is the critical depth; and B_c is the critical top width. If the losses are desired, they can be computed from the equation of specific energy:

$$h_{v1} + H = d_3 + h_{v3} + h_e + h_i \tag{34}$$

in which h_e is the entrance loss, and h_i is the impact loss. Returning to a case of a rectangular channel of uniform width, with vertical upstream face of the weir discussed by the authors: $P_0 = 0$, and Equation (30) may be re-written as follows:

$$\frac{H^2}{2} + 2\,d_1\,h_{v1} = \frac{d_3^{\,2}}{2} + 2\,d_3\,h_{v3} \tag{35}$$

For the flow from a rectangular reservoir, $h_{v1} = 0$, and Equation (35) becomes:

$$\frac{H^2}{2} = \frac{d_3^{\,2}}{2} + 2\,d_3\,h_{v3} \tag{36}$$

Since, for this case, $h_{v3} = n\,d_3 = 0.5\,d_3$, by Equation (31), after substituting in Equation (36):

$$d_3 = \sqrt{\tfrac{1}{3}}\,H = 0.577\,H \tag{37}$$

It may be noted that for an ideal flow without losses, $d_3 = 0.667\,H$.

The losses of entrance and impact corresponding to $d_3 = 0.577\,H$ will be, by Equation (34), $h_e + h_i = H - 0.577\,H - 0.288\,H = 0.135\,H = 0.468\,h_{v3}$, or 47% of the velocity head of velocity on the weir. In the case discussed by the authors, due to velocity of approach, the depth d_3 varied between 0.51 H and 0.55 H and losses varied between 0.30 h_{v3} and 0.07 h_{v3}.

A practical problem arises of determining the discharge from a reservoir of irregular shape. In this case the simplest procedure is to estimate the impact and entrance loss as a percentage of h_{v3}, depending on entrance conditions, and solve for d_3 from Equation (34). The discharge then may be computed either by the formula, $Q = A_3\,V_3$, or by the weir formula, $Q = C\,A\,\sqrt{H}$, which becomes $Q = C\,L\,H^{1.5}$ for a rectangular weir. The value of C may be found by Equation (18), or by the expression:

$$C = (\tfrac{2}{3})^{1.5} \frac{\sqrt{g}}{\left(1 + \frac{p}{3}\right)^{1.5}} \qquad (38)$$

in which

$$p = \frac{h_e + h_i}{h_{v3}} \qquad (39)$$

is the ratio of loss in terms of velocity head and velocity on the weir. Equation (38) is obtained from the expression

$$C = \frac{A_3\,V_3}{L\,H^{1.5}} \qquad (40)$$

after substitution of $A_3 = d_3\,L$ and $V_3 = \sqrt{2\,g\,h_{v3}}$, with $h_{v3} = \frac{d_3}{2}$ and d_3 expressed through H and p from Equation (34). The force P_0 can be computed then from Equation (31) if desired and compared with actual existing conditions.

The value of P_0 is maximum for zero losses and becomes zero for the maximum loss obtainable for the given shape of the reservoir and weir. For the rectangular reservoir, the ratio of $\frac{P_0}{P_1}$, determined from Equations (31) and (34), is equal to 0.333 for zero loss and is equal to 0.000 for a loss ratio $p = 0.468$. For a sharp-crested weir the ratio $\frac{P_0}{P_1}$ was found by Charles W. Harris,[22] M. Am. Soc. C. E., to be equal to 0.222. For a reservoir of irregular shape the ratio $\frac{P_0}{P_1}$ will change, but the trend of its variation will remain the same.

In closing, the writer wishes to remark that the generally adopted use of the coefficient of discharge C, without proper correlation with the energy conditions of the flow, does not help to explain the mechanics of the latter. For instance, what does coefficient $C = 2.85$ mean if it is not related through definite entrance and impact losses to the physical condition of the entire entrance and not just to the "entrance lip"? This is exactly what has been done in the past—the coefficients C were assumed arbitrarily, based only on the shape of the "entrance lip" and for a rectangular weir, whereas, in practice, weirs and approach channels are encountered in any form. The work done by the authors will enable designers to correlate the losses occurring in transition to the coefficient of discharge C in a more rational manner.

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[22] "An Analysis of the Weir Coefficient for Suppressed Weir 5," by Charles W. Harris, *Bulletin No. 22*, Univ. of Washington, November 15, 1923.

JOHN W. HACKNEY,[23] JUN. AM. SOC. C. E. (by letter).—In Equation (18) the authors define the relationship between the weir coefficient (Ω), the head on the weir (H), and the height of the weir face (d_4). If, as the authors suggest, K may be assumed to be a constant and equal to 2.00, this formula may be reduced in the following manner:

$$\Omega = \left[\frac{g}{2}\left(\frac{4-1}{4}\right)\left(\frac{d_4 + H}{2\,d_4 + H}\right)\right]^{0.5} = \left[\;12.1\left(\frac{1 + H/d_4}{2 + H/d_4}\right)\right]^{0.5} \quad \ldots (41)$$

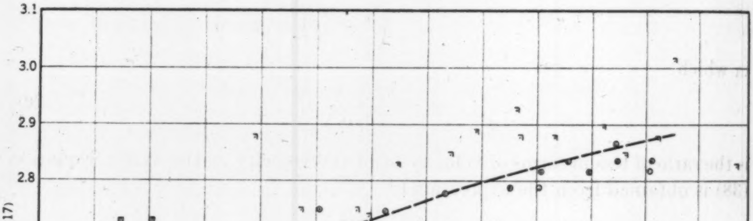

FIG. 12

In Equation (41), it is apparent that Ω is a function of $\frac{H}{d_4}$ only. Plotting the authors' test results from Table 2 in the form Ω versus $\frac{H}{d_4}$, Fig. 12(a) is

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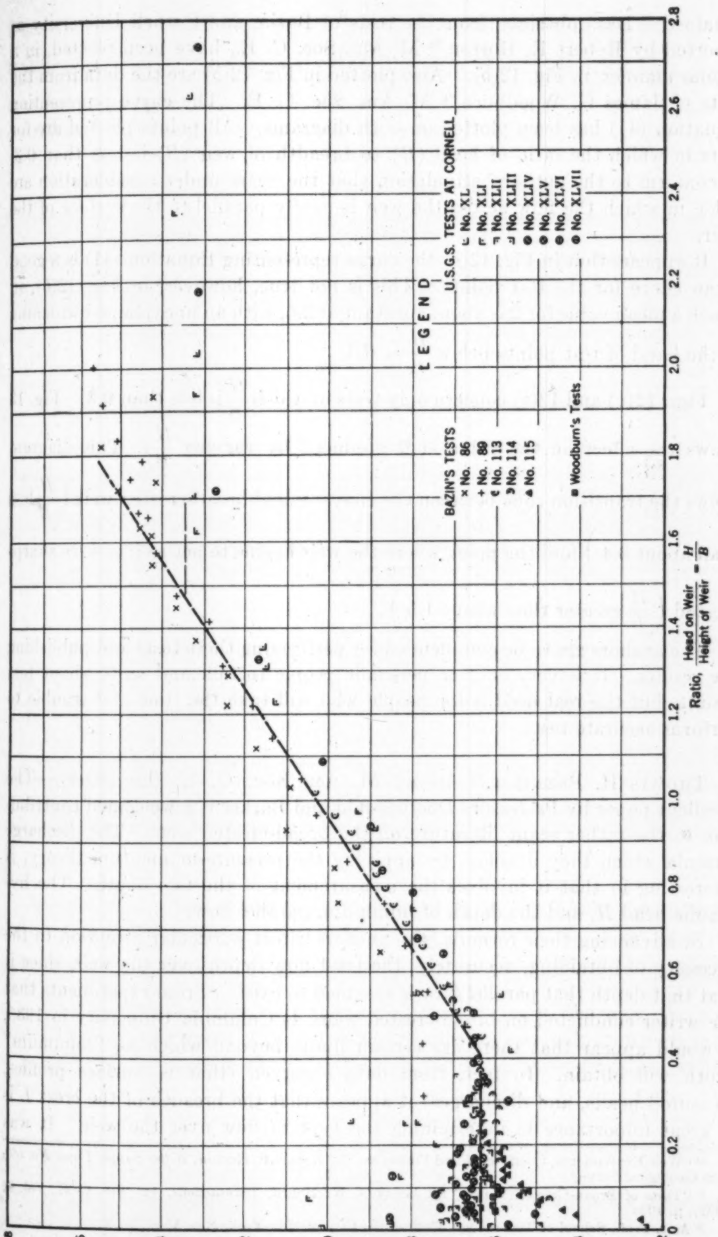

FIG. 13.—DISCHARGE COEFFICIENTS FOR SHARP-CORNERED, BROAD-CRESTED WEIRS AND FOR VARYING VALUES OF $\frac{H}{B}$

obtained. Data obtained from the tests of Bazin, and Cornell University as reported by Robert E. Horton,[24] M. Am. Soc. C. E., have been plotted, in a similar manner, in Fig. 12(*b*). Also plotted in Fig. 12(*b*) are the data from the tests of James G. Woodburn,[25] M. Am. Soc. C. E. The curve representing Equation (41) has been plotted on both diagrams. All points plotted are for tests in which the ratio of head (H) to breadth of weir (B) is less than 0.3, to conform to the authors' stipulation that the weirs under consideration are those in which the flow across the weir is nearly parallel to the surface of the weir.

It appears that in Fig. 12(*a*) the curve representing Equation (41) is a good mean curve for the test points. This is not true, however, in Fig. 12(*b*), in which a mean value for Ω is about constant at 2.6, with an unexplained widening of the band of test points below $\frac{H}{d_4} = 0.3$.

Figs. 12(*a*) and 12(*b*) concern only tests in which $\frac{H}{B}$ is less than 0.3. Fig. 13 shows the effect on the value of Ω produced by varying $\frac{H}{B}$. This diagram shows the transition zone between the conventional broad-crested weir $\left(\frac{H}{B}\text{ less than about } 0.4\right)$ and the point where the weir begins to act as if it were sharp-crested $\left(\frac{H}{B}\text{ greater than about } 1.5\right)$.

The authors are to be commended for performing these tests and publishing the results. It is very easy to persuade people to talk and write about test results, but the real need is for people who will take the time and trouble to perform accurate tests.

Thomas H. Prentice,[26] Assoc. M. Am. Soc. C. E. (by letter).—The excellent paper by Professors Doeringsfeld and Barker is a welcomed contribution to the rather scant literature on the broad-crested weir. The discharge formula which they develop, by applying the pressure-momentum theory, is interesting in that it involves the measurement of the two depths: The hydraulic head H, and the depth of minimum, parallel flow.

In advancing their formula, the authors direct particular attention to the necessity of obtaining, accurately, the least flow depth over the weir, since it is at that depth that parallel flow is assumed to exist. From experiments that the writer conducted on broad-crested weirs at Columbia University in 1935, it would appear that there are certain limits beyond which no "minimum" depth will obtain. In fact, from data observed (that is, surface profiles, measured heads, and discharges) it appears that the breadth of the crest L is of great importance in determining the type of flow over the weir. It was found that if the breadth of crest L and the hydraulic head H are related by writing the ratio $\frac{L}{H}$, flow approximating the parallel pattern exists only when that ratio is greater than 5. For values less than this the flow was distinctly curvilinear over the entire breadth of the crest. Therefore, the proper application of the pressure-momentum theory should be in those cases in which $\frac{L}{H}$ has a value greater than 5.

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[24] "Weir Experiments, Coefficients and Formulas," by Robert E. Horton, *Water Supply Paper No. 150*, U. S. Geological Survey.

[25] "Tests of Broad-Crested Weirs," by James G. Woodburn, *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 405.

[26] Asst. Prof., School of Technology, Coll. of the City of New York, New York, N. Y.

TABLE 5.—RECOMPUTATION OF DATA IN TABLES 1 AND 2

Head, H, in feet	$K = \frac{H}{d_3}$	DISCHARGE, IN CUBIC FEET PER SECOND			Ratio, $\frac{L}{H}$	Head, H, in feet	$K = \frac{H}{d_3}$	DISCHARGE, IN CUBIC FEET PER SECOND			Ratio, $\frac{L}{H}$
		Q (observed)	Ω_0 (observed)*	Ω_e (computed)†				Q (observed)	Ω_0 (observed)*	Ω_e (computed)†	
(1)	(2)	(3)	(4)	(5)	(6)	(1)	(2)	(3)	(4)	(5)	(6)
(*a*) MINNESOTA TESTS (SEE TABLE 1(*a*))											
0.117	1.95	0.170	2.53	2.59	19.2	0.355	1.97	0.955	2.69	2.76	6.3
0.128	1.83	0.193	2.50	2.63	17.6	0.430	1.96	1.280	2.70	2.81	5.2
0.156	1.84	0.270	2.61	2.66	14.4	0.480	1.92	1.560	2.79	2.85	4.7
0.230	1.92	0.500	2.70	2.69	9.8	0.528	1.88	1.820	2.82	2.89	4.3
0.270	1.93	0.647	2.75	2.72	8.3	0.552	1.94	1.973	2.86	2.88	4.1
(*b*) WASHINGTON TESTS (SEE TABLE 1(*b*), HEIGHT, 0.262 FT)											
0.053	1.29	0.027	2.66	2.39	21.1	0.207	1.83	0.227	2.89	2.85	5.4
0.090	1.49	0.065	2.89	2.68	12.4	0.226	1.86	0.262	2.93	2.86	5.0
0.103	1.64	0.079	2.87	2.73	10.9	0.228	1.90	0.2615	2.88	2.85	4.9
0.124	1.70	0.100	2.75	2.76	9.0	0.245	1.90	0.291	2.88	2.87	4.6
0.132	1.86	0.110	2.75	2.74	8.5	0.268	1.88	0.335	2.90	2.90	4.2
0.151	1.92	0.1348	2.76	2.75	7.4	0.279	1.86	0.350	2.85	2.92	4.0
0.157	1.84	0.142	2.74	2.78	7.1	0.295	1.88	0.385	2.88	2.94	3.8
0.170	1.77	0.165	2.83	2.82	6.6	0.302	1.90	0.418	3.02	2.92	3.7
0.186	1.86	0.181	2.71	2.82	6.0	0.334	1.90	0.455	2.83	2.95	3.3
0.195	1.82	0.204	2.84	2.84	5.7	....	....	....	....	....	....

* From Equation (42). † From Equation (18).

Another assumption of the pressure-momentum theory is that the pressure distribution on the upstream contracted section is hydrostatic. That observation is an approximation, permissible only within certain limits of $\frac{H}{d_4}$, in which d_4 is the height of the weir. At the upstream entrance to the weir, the manometers which the writer used to record pressures on the weir crest indicated in a very striking manner that there was a region of under-pressure, at the entrance, by a very decided dip in the pressure profile. In this region the flow is most sharply curvilinear, the surface profile being convex upward. Pressure profiles plotted above the crest showed other regions of curved flow, being above the surface profile when the flow was concave upward (pressures greater than hydrostatic) and below the surface profile when the flow was convex upward (pressures less than hydrostatic).

Manometer readings for the piezometer connections on the vertical upstream face of the weirs showed that the pressures were substantially hydrostatic for the lower points on the face, but that there was considerable deviation of the pressure line inward toward the weir face in the region adjacent to the crest level; such departure from the true hydrostatic pressure was due to the velocity head of approach of the oncoming flow.

TABLE 6.—VALUES OF C, COLUMBIA TESTS; $d_4 = 0.667$ FT AND $b = 0.50$ FT

Head, H, in feet	Observed discharge, Q, in cubic feet per second (1)	Coefficient, C, in Equation (44) (observed) (2)	Ratio, $\frac{L}{H}$ (3)	Observed discharge, Q, in cubic feet per second (1)	Coefficient, C, in Equation (44) (observed) (2)	Ratio, $\frac{L}{H}$ (3)
	(a) LENGTH OF WEIR $L = 2.00$ FT			(b) LENGTH OF WEIR $L = 3.00$ FT		
0.100	0.086	2.70	20.0	....	....	....
0.200	0.243	2.69	10.0	0.244	2.70	15.0
0.300	0.449	2.69	6.67	0.452	2.71	10.0
0.400	0.698	2.70	5.00	0.703	2.72	7.5
0.500	0.986	2.71	4.00	0.995	2.73	6.0
0.600	1.312	2.72	3.33	1.325	2.75	5.0

In advancing new formulas resulting from tests, the question is generally raised as to whether there is a simplification and advantage over the formulas already in general use. Being interested in comparing the results of his tests with those obtained by the authors, the writer recomputed some of the data appearing in that paper, as shown in Table 5. The values of Ω in Columns (5) are discharge coefficients computed from Equation (18). The values marked as observed are determined from

$$Q = \Omega_0 \, b \, H^{1.5} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (42)$$

Both sets of values were based upon the values of the discharge, Q, given in the authors' data. There is also presented in Table 6 values of the respective

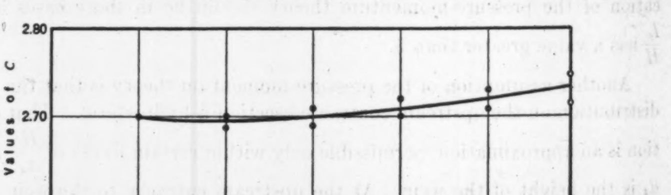

FIG. 14:—PLOTTED VALUES OF C VERSUS H IN FRANCIS' FORMULA, EQUATION (43)

discharge coefficient, C, using Francis' formula for data obtained in the Columbia tests:

$$Q = M \times \tfrac{2}{3} \times b \times \sqrt{2\,g} \left[\left(H + \frac{v^2}{2\,g} \right)^{1.5} - \left(\frac{v^2}{2\,g} \right)^{1.5} \right] \ldots\ldots (43)$$

in which

$$M \times \tfrac{2}{3} \times \sqrt{2\,g} = C \ldots\ldots\ldots\ldots\ldots\ldots (44)$$

It will be observed that, in general, the value of C in Table 6 is rather steady, approximating the commonly used average, $C = 2.70$ (see Fig. 14).

The writer is in accord with the statement made by the authors that there should be considerable caution in applying their formula in cases where there is curvilinear flow over the entire breadth of the crest. Might not the question be asked whether some of the discrepancies in the values of Ω in the formula offered by Professors Doeringsfeld and Barker (Equation 18) could not be caused by difficulties in obtaining true minimum depths because of sharply curvilinear flow?

Boris A. Bakhmeteff,[27] M. Am. Soc. C. E. (by letter).—An interesting attempt to apply the momentum principle in establishing the interrelations which govern the inflow to a broad-crested weir is presented in this paper. The writer presumes that the analysis is meant to apply to a sharp-edged entrance only, for in this case the vein jumps the crest and produces a roller such as that in Fig. 15. "Separation" and the brusque change of flow forms are accompanied by a substantial local entrance loss. The mechanical premises resemble those in a hydraulic jump or in the sudden expansion of a closed conduit. The tangential friction plays a negligible rôle and can be omitted from consideration, while the intense energy losses, due to concentrated local turbulence, are accounted for by the pressure differences in the controlling sections. Experience shows that, notwithstanding the approximate character of the analysis, the momentum principle in such cases expresses the bulk interrelations in quite a satisfactory manner. The instance of the sharp-edged, broad-crested weir, investigated by the authors, offers one more proof of the accord between theory and experiment. K. Woycicki applied[28] the momentum principle under very similar conditions to investigate the roller-covered submerged efflux from under a sluice.

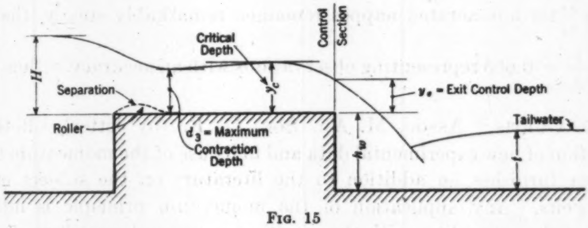

Fig. 15

The authors' findings center around Equation (18), which constitutes a "discharge coefficient," the numerical value of which can be determined by inserting into the formula the two easily measurable quantities, H and d_3. It appears that in this manner Equation (18) can be applied usefully for approximate bulk appraisals, such as in the design of structures. On the other hand, neither Equation (18), nor any other customary formula derived from the momentum principle, possesses the degree of accuracy required for precise metering.

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[27] Prof., Civ. Eng., Columbia Univ., New York, N. Y.

[28] "Wassersprung, Deckwalze und Ausfluss unter einer Schutze," by K. Woycicki, *Verlag* der Polnischen Akademie der Technischen Wissenschaften, Warsaw, 1931.

It is questionable whether the very thought of using the entrance section of the broad-crested weir as a metering device is expedient, in general, particularly since Hunter Rouse,[29] Assoc. M. Am. Soc. C. E., disclosed the advantages of the exit section as a possible control, pointing out that, for fully aerated nappes, the ratio $\frac{y_e}{y_c}$ (Fig. 15) remains consistently stable and equal to 0.715. The possible disadvantage in Mr. Rouse's method is the difficulty of providing adequate ventilation for the nappe under field conditions. Therefore, further experiments were conducted by T. B. Robinson, Jun. Am. Soc. C. E., in the Fluid Mechanics Laboratory at Columbia University with the view of establishing whether it was practicable to use the exit section of a broad-crested weir as a control in the presence of unventilated nappes. The results obtained indicate that, notwithstanding the wide change in tailwater depth $\left(\frac{t}{h_w}\text{ up to near }0.9\right)$, the ratio $\frac{y_e}{y_c}$ for non-aerated nappes remained remarkably steady, the average value of $\frac{y_e}{y_c} = 0.665$ representing observations with an accuracy within 1%.

D. D. Curtis,[30] Assoc. M. Am. Soc. C. E. (by letter).—Both in its presentation of new experimental data and in its use of the momentum theorem, this paper furnishes an addition to the literature on the subject of broad-crested weirs. Any application of the momentum principle is interesting. Because of the mutuality, with the resulting equational cancellation, of forces involved, the principle commonly known as the conservation of momentum is one of the neatest devices of mechanics. However, as in most such applications in hydraulics, the problem of the paper is one of continuous action of forces on a body with consequent change in momentum, rather than of the interaction of two bodies on each other, as implied in the momentum-conservation principle.

In cases where conservation of momentum does not apply, full consideration of all forces involved is essential to rigidity. Some years ago, in conducting research on the expanding stream below a particular submerged weir, the writer had occasion to apply the momentum formula in predicting water-surface elevations. The results were gratifyingly successful, although it was not found possible to consider certain of the less important forces.

It is evident that the authors ignored certain items in addition to the mentioned friction loss between the crest and the section of parallel flow. For example, there is a downward acceleration of part of the water. This can be produced only by a reduction of the pressure intensity upward on the water to a value less than the static pressure since there must be a resultant force downward, and the only downward force acting on the water—the gravity pull—is the same in amount as the static pressure. The effect on the derivation of neglecting this pressure reduction is problematical, but it seems likely to influence the assumption of hydrostatic pressure on the upstream face of the weir. Furthermore, the momentum (which is scarcely to be considered a force, but rather a mass-velocity property) is not correctly computed from the average velocity. It will be noted that the error from the use of average velocity for calculating momentum is of the same order as that from using average velocity in computing velocity head. This is one degree less in the velocity exponent than that in computing kinetic energy on the same assumption.

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[29] "Discharge Characteristics of the Free Overfall," by Hunter Rouse, *Civil Engineering*, April, 1936, p. 257.

[30] Prof. of Mechanics and Hydraulics, Clemson Coll., Clemson, S. C.

In regard to this latter point (the effect of use of average velocity) it is possible to see something of the magnitude of the error in the instance of momentum (and velocity head). With uniform velocity variation from zero to V between the vertical limits of the stream, the true momentum would be twice that calculated on the basis of the average velocity. For parabolic distribution from zero to V, the parabola being concave to the axis from which velocity is measured, the true momentum would be 1.35 times the calculated one; with uniform variation from 0.5 V to V the true momentum would be 1.11 times the calculated one; with uniform variation from 0.67 V to V the true momentum would be 1.04 times the computed one. This can be seen to hold possibilities of serious error, dependent upon the velocity distribution. In the case of the theory developed in this paper, it is not likely to be significant.

To afford a comparison with the authors' results, data were recomputed for runs made by the writer on a weir 1.5 ft high, with crest rounded to a 9-in. radius, with upstream face sloped at 40° and tangent to the rounded crest, and with downstream face similarly tangent to the crest circle but adjustable for slope. The main purpose was quite different from that of this paper, and conditions under which the weir was operated in many runs were not suitable to direct comparison with the authors' work. For such as were made under a comparable regimen of flow, values of K in Equation (16) were found to range close to 1.90. The fact that the value of K obtained under decidedly different conditions closely approached that found by the authors gives weight to the belief that the items disregarded are unimportant in their effect. In any event the size of Ω is not particularly sensitive to the value of K. Calculations show that reducing K from 2 to 1.97 would make a difference of less than 1% in Ω; reduction to 1.94 would correspondingly make a difference of less than 2%; and further reduction to 1.90 would make less than 2.5% difference from that obtained for $K = 2$. Therefore, the use of the value 2 appears warranted.

In spite of the defects pointed out, the paper is interesting as a new approach to the problem of the broad-crested weir. It is hoped that the work will be continued and expanded. For instance, rounding of the upstream edge of the crest would make for greater stability in the water surface with probably even closer agreement of results with theory.

Carl Rohwer,[31] Assoc. M. Am. Soc. C. E. (by letter).—The possibilities of broad-crested weirs as measuring devices have long been recognized by engineers, and extensive tests have been made to determine their characteristics. It was assumed that by measuring the critical depth the discharge over this weir could be determined easily and accurately because of the simple relation existing between the critical depth and the discharge.

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[31] Irrig. Engr., Div. of Irrig., SCS, U. S. Dept. of Agriculture, Colorado Experiment Station, Fort Collins, Colo.

A series of tests by James G. Woodburn,[3] M. Am. Soc. C. E., showed, however, that the location of the critical depth varied with the shape of the weir and with the head, and that consequently the critical depth could not be used as the basis in determining the discharge. R. L. Parshall, Assoc. M. Am. Soc. C. E., conducted tests on several types of broad-crested weirs and found similar results.[22]

The application of the pressure-momentum theory to the flow over broad-crested weirs is another attempt to derive a formula suitable for measuring the flow over this type of weir. The formula derived by the author (Equation (16)) is not so simple as the critical-depth formula, but since it involves all the factors influencing the flow it gives a direct solution of the discharge. In the formula, K is substituted for the ratio of the head H, on the weir, to the depth d_3, on the top of the weir in the zone of parallel flow. If K is known, then the measurement of the single head H determines the discharge. Since the tests made by the authors show that K is not constant (Table 1, Column (3)), it seems to the writer that it would be desirable to make direct measurements of both H and d_3, thereby eliminating the uncertainty as to the value of K. It is true, as stated by the authors, that small variations in K do not seriously affect the results (Table 2, Columns (5), (6), (11), and (12)), but, in case the tailwater were high enough to retard the flow over the weir, there would be a material change in K and probably also in the discharge computed by the formula unless the true value of K was used. Since the derivation of Equation (16) is a general one (in that the degree of submergence is not a factor in the accuracy of the analysis by the pressure-momentum theory), it seems to the writer that this is an important feature of the formula and that tests should be conducted to find the degree of submergence permissible without affecting the results.

The computed curves and the experimental results shown in Fig. 9, for weirs with heights of 1.424, 1.256, 0.965, 0.781, and 0.313 ft, indicate that, with a few exceptions, the observed discharges are consistently greater than the computed discharges. This condition might result from an error caused by the method of measuring the gage heights or from some error in the assumptions which have to be made in applying the pressure-momentum theory to the problem of flow over broad-crested weirs. Whatever the cause of the discrepancy in the results, the differences are so small that a slight correction of the formula in the nature of an empirical constant or coefficient would apparently bring the observed and computed results into very close agreement.

The profiles of the water surfaces and the pressure heads in Fig. 10 show that piezometer readings may differ materially from the true water depths. In fact, the differences in zones where the water-surface profiles have a pronounced curvature are so great that piezometer readings would give an entirely erroneous indication of the true water depth. Where parallel flow exists, the water surface and the piezometer surface are nearly the same. It is interesting to note that Professor Bakhmeteff in his discussion of Professor Woodburn's paper[15] demonstrated that when the surface of the water was convex the piezometric surface would be below the water surface and that when the water surface was concave the piezometric surface would be above the water surface. This is shown clearly in Fig. 10. At points of inflection in the water-surface curve the water is not changing direction and consequently the two surfaces should agree. This fact also is confirmed by the profiles in Fig. 10.

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[22] Discussion by R. L. Parshall of "Tests of Broad-Crested Weirs," by James G. Woodburn, *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 440.

The broad-crested weir cannot take the place of the sharp-crested weir in the laboratory, and in the field it has some of the same limitations that restrict the use of the sharp-crested weir. The principal objection is that the pool formed in front of the weir is a natural settling basin for debris. The accumulation of material in the pool reduces the height of the weir and also increases the velocity of approach; consequently the formulas generally applied give erroneous results. The formula derived by Messrs. Doeringsfeld and Barker for broad-crested weirs is subject to the same criticism because the height of the weir is one of the factors in the formula, and obviously if the height has been reduced by the accumulation of silt, the formula will not give correct results unless the actual height of the weir is determined each time before applying the formula. It is believed by the writer that control sections, such as the Parshall flume,[33] are more satisfactory than weirs for use in streams and canals because there is no obstruction in the bottom of the channel and consequently no tendency for silt to deposit under ordinary conditions.

JOHN HEDBERG,[34] ASSOC. M. AM. SOC. C. E. (by letter).—On a slope steep enough to warrant the use of the cosine of the angle of inclination the pressure is definitely not equal to the weight of the column of liquid above it.' A more exact value is considerably less than this.[35] However, the entrainment of air that usually accompanies steep-slope flow makes each of the terms of Equation (9) too uncertain for use. The fact that the authors apply their theory only to the special case of horizontal flow makes everything come out all right so that the final Equation (16) is correct.

The authors should present the pressures on the downstream face of the weir and extend the surface profile some distance downstream. These data will be very valuable for future investigations.

Another point that deserves attention is the rôle of the hanging eddy along the upstream edge. The authors are correct in saying that curvature of flow makes the simple critical-depth formula inaccurate, but they failed to note that the eddy hanging along the sharp crest acts like an additional height of weir. The flow in the central portion of the weir is made subcritical by the flow down the sloping rear face of the eddy region. Possibly the fluctuations in the size of this vortex space are the major factors in the variations in the value of K. Further study of this point would seem desirable.

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[33] "The Improved Venturi Flume," by R. L. Parshall, *Transactions*, Am. Soc. C. E., Vol. 89 (1926), pp. 841–880.

[34] Asst. Prof., Civ. Eng., Stanford Univ., Stanford University, Calif.

[35] "Fluid Mechanics for Hydraulic Engineers," by Hunter Rouse, Assoc. M. Am. Soc. C. E., McGraw-Hill Book Company, Inc., New York and London, 1st Ed., 1938, p. 281.

H. A. Doeringsfeld,[36] Esq., and C. L. Barker,[37] Assoc. M. Am. Soc. C. E. (by letter).—In closing the discussion on their paper the writers wish to express their appreciation to the men who have so kindly discussed the results. Many valuable suggestions have been made and it is hoped that the information contained in the paper and the discussions will be of value in further study.

The observation made by Mr. Hackney as to the range of the ratio of $\frac{H}{B}$ is important. It emphasizes the importance of being able to obtain the position of "nearly parallel" flow. This observation agrees closely with the value given by Professor Prentice of $\frac{B}{H} = 5$ (or $\frac{H}{B} = 0.2$). He states that parallel flow existed only when $\frac{H}{B}$ was less than about 0.2.

The suggestion by Professor Prentice that some of the discrepancies in the values of Ω might be caused by the difficulties in obtaining the true minimum depths is undoubtedly correct. To read the water surface elevation to an accuracy greater than 0.001 ft is quite impossible; yet errors in d_3 of that size might affect the value of K appreciably, especially at the low heads.

Professor Bakhmeteff has emphasized quite well what the writers tried to state in cautioning about the use of Equation (18).

In their studies the writers considered the possibility of effect of the downward acceleration of the water as suggested by Professor Curtis. However, not knowing just how to account for the effect of this force in the computations it was neglected, as was also friction; and a study was made to determine what results could be obtained without considering these effects.

The fact that the true momentum cannot be computed by the use of the average velocity has been mentioned by several writers. Professor Curtis states how much in error the momentum, computed by average velocity, might be. A detailed study of the velocity distribution would be necessary to determine, accurately, the size of the errors that develop through the use of the average velocity.

Mr. Rohwer states that the broad-crested weir is subject to the same limitations as the sharp-crested weir in the field because of the settling basin for debris. However, if the crest height is quite large, the effect of small changes of crest height due to the deposit of debris is slight and quite satisfactory results can be obtained. Of course, a large layer of sediment behind the weir will affect the results.

On the Minnesota weir tests, the water-surface profile on the downstream end of the weir was taken and these data were carefully studied but not published. Professor Hedberg suggests that they might be of value. However, the surface was so rough that its mean plane was difficult to obtain. In the tests mentioned, the tailwater surface was raised from the lowest position possible to a position at which the "nearly parallel flow section" was forced upstream. Between these two positions the tailwater surface elevation had no effect on the upstream head; but when the "nearly parallel flow position" was forced upstream, the head above the weir was changed.

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[36] Asst. Prof., Mathematics and Mechanics, Univ. of Minn., Minneapolis, Minn.

[37] Asst. Prof., Hydr. Eng., Coll. of Eng., State Coll. of Washington, Pullman, Wash.

Professor Hedberg also mentions the interesting fact that the eddy overhanging the weir increases its effective height. This is an important observation and should be studied, as he suggests.

Messrs. Stevens, Curtis, and Hedberg mention the fact that if the slope of the crest is made other than zero, the equations will be changed. This is true, of course. The crest was made level in all tests to keep all equations as simple as possible. As they mention, however, if the crest had a slope the equations could easily be developed from theory. It is well that they stressed the point that the slope was zero so that some attempting to use the expression would not use it in error.

Mr. Wilm states that "Throughout the paper, apparently, the assumption is made that parallel flow will occur over the level weir surface, except where the flow is influenced by end conditions." This statement is scarcely correct. Certainly flow cannot exist in normal conditions with the bed of a channel horizontal and the water surface parallel to the bed. There must be a slope provided either to the surface, or the bed, or both. Perhaps the writers are in error in the use of the words "nearly parallel," for there is no degree of parallelism. In the "Conclusion," the statement was made by the writers: "The accuracy of the formulas is to be questioned if the breadth of the weir is so small that the water surface does not become parallel to the surface of the weir, or nearly so." This sentence meant to imply that, for a very short distance, if the stream lines could be considered nearly parallel to the weir surface then in that space, d_3 could be measured. In the case of raising the tailwater mentioned, as long as the position of measurement of d_3 was not shifted, there was no effect on the formula, and certainly, in the extreme high position of the tailwater, one could not consider flow, where the water surface at the d_3 position was concave upward, as a position of parallel flow.

A sharp-crested weir was used only because its very sharpness precludes the need for giving a radius of curvature. Certainly many weirs of different crest profiles could be studied, and perhaps better results obtained, since, by proper design, the vena contracta could be eliminated.

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Paper No. 2118

MODEL TESTS, BRIDGE PIER SUPPORTED ON LONG STEEL PILES

BY THOMAS F. COMBER, JR.,[1] M. AM. SOC. C. E., AND
JOHN M. COAN, JR.,[2] JUN. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. JACOB FELD, AND THOMAS F. COMBER, JR., AND JOHN M. COAN, JR.

SYNOPSIS

Many theories[3] on the stress analysis of groups of long, unsupported piles have been advanced without adequate substantiation by test data. This paper gives the results of tests on a model of a proposed deep-water bridge pier supported on long, tubular steel piles. The model conformed to the prototype except for the neglect of the restraining action of the foundation material. Pile stresses and pier deflections were measured in the model from which corresponding values may be determined in the prototype.

These test data are submitted in the hope that they may provide a check for one of the previously advanced theoretical analyses, or may lead to a new treatment, or, in any case, may add to the knowledge of the structural behavior of such pile groups.

INTRODUCTION

In many locations where bedrock is heavily overlaid with alluvial deposits and deep water, important river crossings are prohibited by the high cost of pier construction resulting from the use of deep caissons or cofferdams. This economic disadvantage might be overcome by the use of piers consisting of a concrete pier cap, supported on groups of long piles, with the elevation of the base of the cap only slightly below mean low water. In such a pier, construction difficulties would be simplified greatly and the quantities of materials much reduced, thus effecting large economies.

The pier which served as a prototype for the test model is shown in Fig. 1. This pier was selected for these tests because of its contemplated use in an actual bridge design. Strains and deflections were measured in the model to determine the probable stresses and deflections in the prototype.

NOTE.—Published in June, 1940, *Proceedings*.

[1] Associate Prof. of Civ. Eng., Civ. Eng. Dept., The Johns Hopkins Univ.; Baltimore, Md.

[2] Stress Analyst, The Glenn L. Martin Co., Middle River, Md.

[3] See various theories in "Design of Pile Foundations," by C. P. Vetter, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 104 (1939), p. 758.

THE MODEL

The model was made in accordance with the principles of similitude.[4] In Fig. 2(*c*), the numerals in parentheses indicate the unrestrained lengths of the piles from cap to base. The companion values, not in parentheses, are the lengths of the piles from the base to their theoretical point of intersection. The approximate length-reduction factor was set by considerations of economy, convenience, etc., to be about forty, and it was established definitely by the available commercial sizes of steel tubing. The contemplated pier design called for piles of steel tubing with an outside diameter of 30 in., a shell thickness of $\frac{7}{16}$ in., and an approximate length of 200 ft. The most convenient tubing for use in the model had an outside diameter of 0.7565 in. and a thickness of 0.030 in. The inside and outside diameters of the ten piles used were mea-

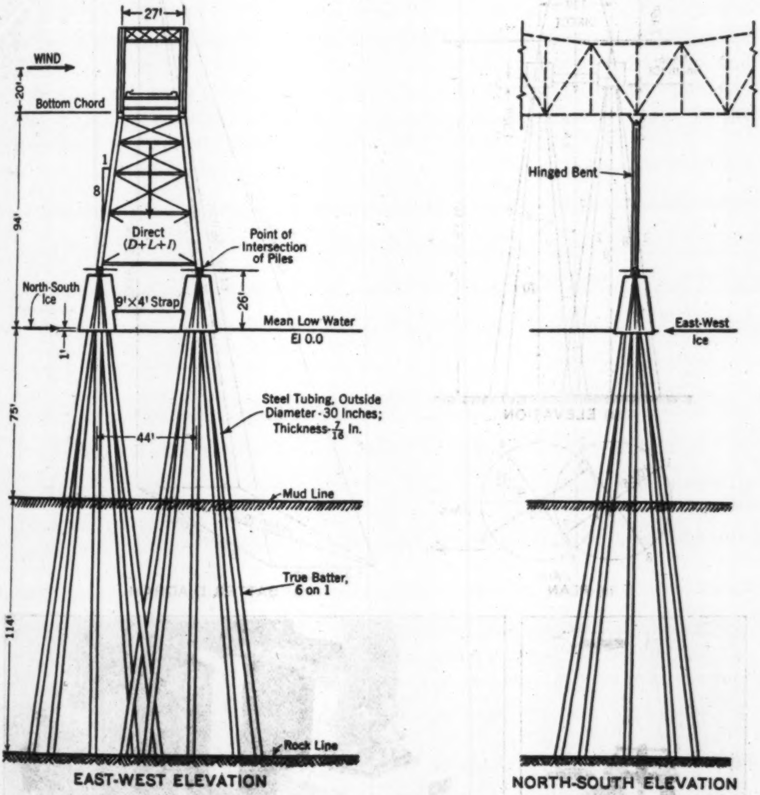
27'
WIND
20'
Bottom Chord
1
8
94'
Direct (D+L+I)
Point of Intersection of Piles
North-South Ice
9'×4' Strap
26'
Mean Low Water
El 0.0
1'
75'
Steel Tubing, Outside Diameter-30 Inches; Thickness-7/16 In.
44'
Mud Line
True Batter, 6 on 1
114'
Rock Line
EAST-WEST ELEVATION
Hinged Bent
East-West Ice
NORTH-SOUTH ELEVATION

FIG. 1.—VIEWS OF PROTOTYPE

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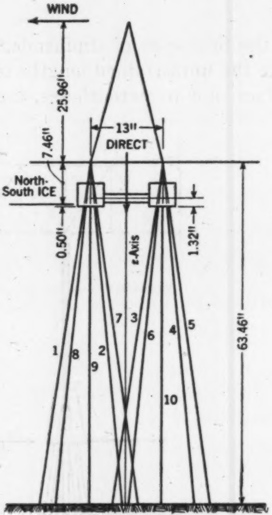
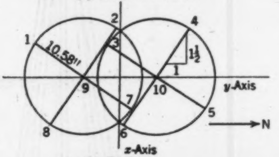
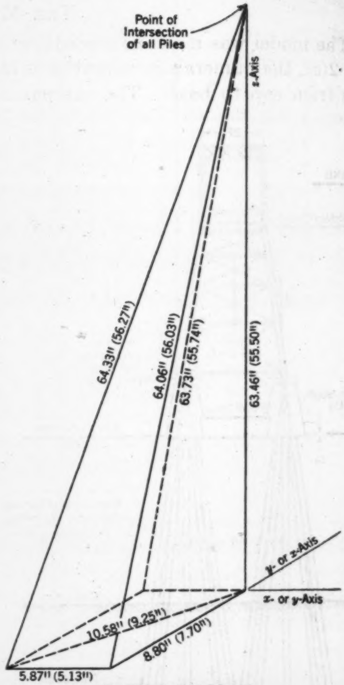
[4] "Similitude Requirements in Model Design," by Roy W. Carlson, Assoc. M. Am. Soc. C. E., *Engineering News-Record*, August 23, 1934.


WIND
25.96"
7.46"
13"
DIRECT
North-South ICE
0.50"
z-Axis
1.32"
63.46"
1
8
9
2
7
3
6
4
5
10

(a) ELEVATION


1
2
3
4
10.58"
9
10
7
8
6
5
1
1½
y-Axis
N
z-Axis

(b) PLAN


Point of Intersection of all Piles
z-Axis
64.33" (56.27")
64.06" (56.03")
63.73" (55.74")
63.46" (55.50")
y- or z-Axis
z- or y-Axis
10.58" (9.25")
8.80" (7.70")
5.87" (5.13")

(c) BATTER DIAGRAM

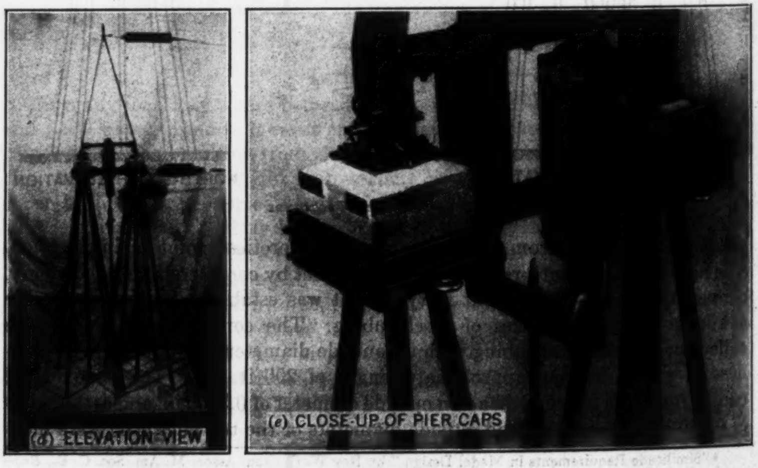

(d) ELEVATION VIEW
(e) CLOSE-UP OF PIER CAPS

FIG. 2.—VIEWS OF MODEL

sured accurately by calipers, and these values represent average values for the entire group.

The length-reduction factor λ is thus equal to the ratio of the mean diameters, or $\lambda = \frac{29.5625}{0.7265} = 40.69$. The force-reduction factor is therefore $\lambda^2 = 40.69^2 = 1{,}655.7$. Two of the three linear dimensions could be adjusted to the length-reduction factor, but the shell thickness dictated by geometric similitude could not be obtained and consequently the measured strains and deflections will have to be interpreted in terms of the required shell thickness. The important constants of a pile cross section in the prototype and in the theoretical and actual models are shown in Table 1.

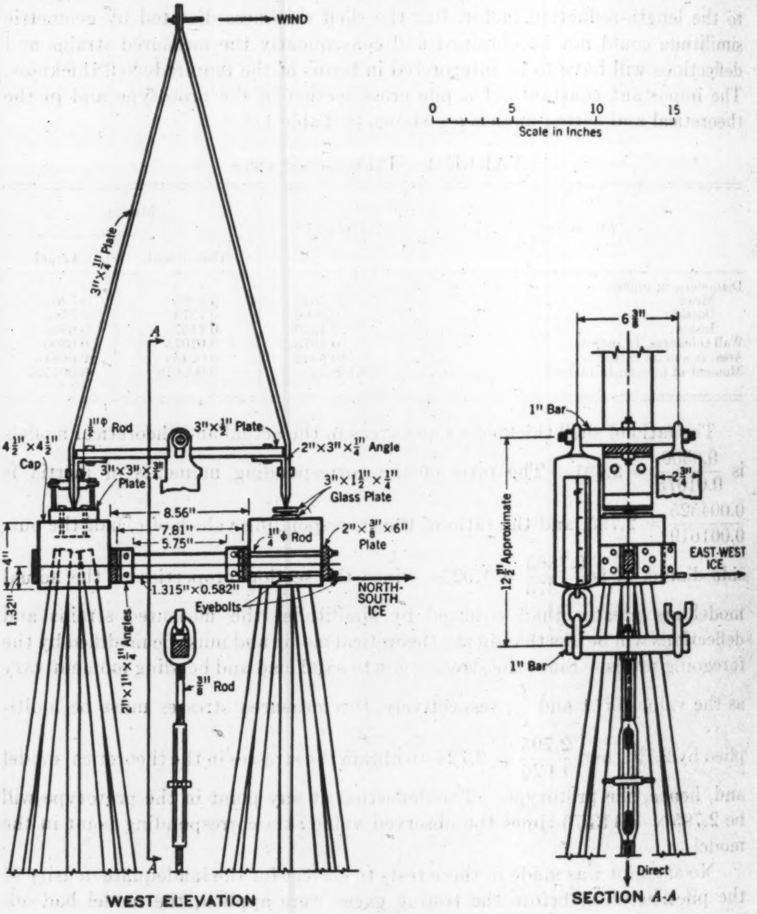
TABLE 1.—Pile Constants

Properties	Prototype	Model	
		Theoretical	Actual
Diameters, in inches			
Mean	29.5625	0.7265	0.7265
Outside	30.0000	0.7373	0.7565
Inside	29.1250	0.7157	0.6965
Wall thickness, in inches	0.4375	0.01075	0.0300
Area, in square inches	40.6322	0.02454	0.06847
Moment of inertia, in inches^4	4,439.74	0.001619	0.004525

The ratio of shell thicknesses and areas in the actual and theoretical models is $\frac{0.0300}{0.01075} = 2.791$. The ratio of the corresponding moments of inertia is $\frac{0.004525}{0.001619} = 2.795$, and the ratio of the corresponding values of c, half the outside diameter, is $\frac{0.7565}{0.7373} = 1.026$. Since the section properties of the actual model are greater than required by similitude, the measured strains and deflections will be less than in the theoretical model and must be modified by the foregoing ratios. Since the stresses due to axial load and bending moment vary as the values of A and $\frac{I}{c}$, respectively, the measured stresses must be multiplied by 2.791, and $\frac{2.795}{1.026} = 2.724$, to obtain the stresses in the theoretical model and, hence, the prototype. The deflection at any point in the prototype will be $2.795\,\lambda = 113.73$ times the observed value at a corresponding point in the model.

No attempt was made in these tests to correct for the inadequate density of the piles because, before the testing gages were applied, the model had adjusted itself to its own weight and that of the permanently attached apparatus (see Figs. 2(*d*) and 2(*e*)). The initial deformations and deflections thus induced were not measured, the gage readings indicating only the effects of the externally applied loads.

The concrete caps were assumed to be inelastic, no account being taken of their internal deformations. The action of the entire structure will depend, however, on the elastic properties of the strut connecting the caps, and its dimensions were adjusted as required by similitude. Had the strut been made of concrete in accordance with the prototype plans, its dimensions would have been $2\frac{5}{8}$ in. by $1\frac{3}{16}$ in. Such a small member would be impracticable and, accordingly, a steel strut of comparable rigidity was used. The ratio of the moduli of elasticity of concrete and steel was approximately fifteen to one and therefore, for an equivalent length, the moments of inertia need be only one fifteenth the value required by similitude. The final section of the steel strut (see Fig. 3) used was 1.315 in. by 0.582 in.


WIND
0
5
10
15
Scale in Inches
3"×1/4" Plate
A
1/2"φ Rod
3"×1/2" Plate
4 1/2"×4 1/2" Cap
2"×3"×1/4" Angle
3"×3"×3/4" Plate
3"×1 1/2"×1/4" Glass Plate
8.56"
7.81"
5.75"
1/4"φ Rod
2"×3/8"×6" Plate
4"
1.32"
1.315"×0.582"
NORTH-SOUTH ICE
Eyebolts
2"×1"×1/4" Angles
3/8" Rod
6 3/8"
1" Bar
2 3/4"
12 1/2" Approximate
EAST-WEST ICE
1" Bar
Direct

WEST ELEVATION

SECTION A-A

FIG. 3.—PIER CAP AND ASSOCIATED APPARATUS

PILE RESTRAINT

The condition that is most difficult to reproduce in the model is the resistance to lateral displacement of the piles offered by the material through which they are driven. This resistance is negligible at the mud line but increases as stiffer materials are encountered at lower elevations. The resistance that successive layers offer to lateral movements seldom varies uniformly, and it is almost impossible to simulate such a resisting medium satisfactorily. Attempts have been made to duplicate such passive action by using similar materials of varying densities and degrees of compaction, as well as purely mechanical devices (such as a series of springs distributed along the pile length and adjusted to represent the supposed variation in resistance). In view of these difficulties, no attempt was made to simulate such restraint.

As constructed, the model represents a system of piles fixed at their bases, restrained by the pier cap, and unrestrained throughout their length, which is, perhaps, the most unfavorable condition of restraint. The stresses so determined should give a fair indication of the maximum stresses to be expected in the prototype.

MEASURING DEVICES

The deflection gages shown in Fig. 4(*a*) were graduated to read in thousandths of an inch and had a maximum travel of about $\frac{3}{8}$ in. Two gages were applied simultaneously at a given point with their axes parallel to the transverse and longitudinal axes of the pier and lying in a horizontal plane. The resulting measurements indicated the movements of that particular point in two perpendicular directions in a horizontal plane.

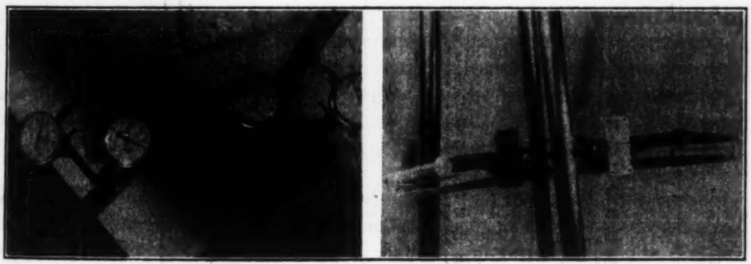

(*a*) (*b*)

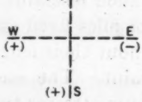
FIG. 4.—TESTING INSTRUMENTS

Two tensometers (see Fig. 4(*b*)), each having a multiplication factor of 1,075, and with 8-in. extensions to further magnify the extremely small strains, were used for the strain measurements. These gages were applied simultaneously at opposite ends of a diameter in either the transverse or longitudinal direction. Readings were taken in this position, and then the two tensometers were applied and read in the perpendicular direction.

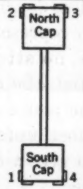
Applied Loads

The direct load (dead, live, and impact) used in the tests was 600 lb, which corresponds to a load on the prototype of 993,420 lb. The applied wind load was 60 lb, being the equivalent of a wind force of 99,340 lb on the trusses. The allowance to be made for ice thrust is quite arbitrary, and in these tests the transverse and longitudinal ice loads were each represented by a force of 120 lb, which corresponds to a thrust on the prototype of 198,680 lb.

TABLE 2.—Location of Test Points



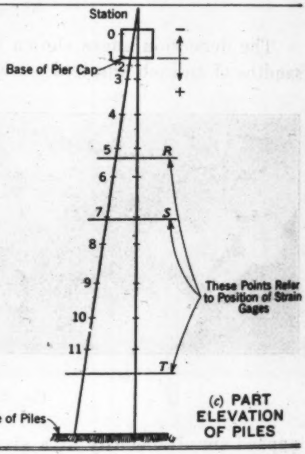
(*a*) SIGN OF DEFLECTIONS



(*b*) PLAN OF PIER CAP

	Distance of Point From Base of Pier Cap, in Inches	
	Battered Pile	Vertical Pile
0[a]		$-3\frac{5}{8}$
1[a]		$-\frac{1}{8}$
Base of Pier Cap		
2	1	1
3	$3\frac{1}{8}$	$3\frac{1}{8}$
4	$8\frac{3}{8}$	$8\frac{1}{4}$
5	$13\frac{1}{2}$	$13\frac{1}{4}$
R	15	$14\frac{3}{4}$
6	$17\frac{3}{4}$	$17\frac{1}{2}$
7	$23\frac{5}{8}$	$23\frac{1}{4}$
S	24	$23\frac{5}{8}$
8	28	$27\frac{5}{8}$
9	$33\frac{3}{4}$	$33\frac{1}{4}$
10	$38\frac{3}{4}$	$38\frac{1}{4}$
11	$43\frac{1}{2}$	$42\frac{7}{8}$
T	47	$46\frac{3}{8}$
Base of Piles		

[a] These Points Are on the Face of the Pier Cap



(*c*) PART ELEVATION OF PILES

The direct load was applied through a steel A-frame (Fig. 3), fixed at one end and freely supported at the other, and the wind and ice loads were applied by means of cables. These load-applying devices were kept constantly alined and were gradually adjusted to their proper tensions by calibrated scales.

Test Points and Loadings

Table 2 indicates the position of the test points. Deflections were measured at stations 0 to 11, and strain measurements were taken at stations R, S, and T. The deflections of the pier cap and the piles due to the four individual effects (direct load, wind load, and two mutually perpendicular ice loads), and to a combination of loads, were observed and recorded. The combination loading represents the simultaneous application of the transverse ice, wind, and direct loads. Deflections and strain measurements were taken as in the case of the individual loads and were compared with the values obtained by adding the individual effects. The object of this comparison was to obtain an idea of the magnitude of the error involved if the secondary effects due to superposition were neglected.

For purposes of brevity, this paper includes only the data and necessary computations for the east-west and the north-south ice loads (east-west is the longitudinal axis of the bridge and north-south the transverse axis). These conditions were selected because the deflections and stresses are of reasonable magnitude and they best illustrate the action of the pier: (1) As two cantilever units (under the east-west ice load); and (2) as a portal (north-south ice load).

Development of the Elastic Curve

The deflections of the various elements of the model were measured for two purposes: (1) To predict the deflections of the prototype pier cap; and (2) to develop the equation of the elastic curve of each pile and, by use of the well-known formula

$$M = E\,I\,\frac{d^2y}{dx^2} \quad \text{.......................} \quad (1)$$

to predict the end moments set up by such deflections. As a check, these end moments will be compared with those determined from the strain measurements.

Any curve, such as the deflection curve of a structural member subject to lateral loading or to applied end moments, may be represented by a cubic equation of the form

$$y = a\,x + b\,x^2 + c\,x^3 \quad \text{.....................} \quad (2)$$

This general equation, although not precise, was used in the analysis because of its simplicity and the accuracy with which the resulting curves agree with the measured deflections. In view of the number of unknown coefficients, Eq. 2 could be solved with a minimum of three sets of coordinates. However, three points would not be sufficient to represent the actual deflected position with any degree of accuracy, and it was decided to use a greater number of points—nine, in this case.

In order to evaluate the coefficients, the coordinates (x, y) of each point are substituted in the general equation, as follows:

$$y_1 = a\,x_1 + b\,x_1^2 + c\,x_1^3 \quad \text{....................} \quad (3a)$$

$$y_2 = a\,x_2 + b\,x_2^2 + c\,x_2^3 \quad \text{....................} \quad (3b)$$

$$y_3 = a\,x_3 + b\,x_3^2 + c\,x_3^3 \quad \text{....................} \quad (3c)$$

$$\cdot\;\cdot\;\cdot\;\cdot\;\cdot\;\cdot\;\cdot\;\cdot\;\cdot\;\cdot\;\cdot\;\cdot$$

$$y_n = a\,x_n + b\,x_n^2 + c\,x_n^3 \quad \text{....................} \quad (3n)$$

If Eqs. 3 are added:

$$\Sigma y = a\,\Sigma x + b\,\Sigma x^2 + c\,\Sigma x^3 \ldots\ldots\ldots\ldots\ldots\ldots \text{(4a)}$$

Furthermore, if Eqs. 3 are multiplied by x_1, x_2, x_3, etc., respectively, and again by x_1^2, x_2^2, x_3^2, etc., the result is two sets of equations which, when added, have the following general form:

$$\Sigma(xy) = a\,\Sigma x^2 + b\,\Sigma x^3 + c\,\Sigma x^4 \ldots\ldots\ldots\ldots\ldots \text{(4b)}$$

and

$$\Sigma(x^2y) = a\,\Sigma x^3 + b\,\Sigma x^4 + c\,\Sigma x^5 \ldots\ldots\ldots\ldots\ldots \text{(4c)}$$

Eq. 2 lacks precision, in that it fails to satisfy a primary condition. By differentiating and setting $x = 0$, $\frac{dy}{dx} = a$, which is obviously incorrect since the slope at the origin is zero, in view of the fact that the pile is fixed at this point. However, this slope is extremely small and Eq. 2 will be used since the corresponding curves fit the given points more closely than those obtained from some equations that satisfy this theoretical condition.

Each pile is assumed to have its maximum deflection, and consequent maximum bending stresses, in a plane which contains that pile and whose horizontal trace is parallel to the direction of the applied load. There are small deflections (Tables 3 and 4) in a direction perpendicular to that of the external loading and, theoretically, these should be resolved with the former in order to arrive at the maximum values indicated by test. However, in all cases of lateral loading, the small deflections that accompany the much larger deflections in the plane of loading are less than 10% of the latter and are neglected. The neglect of these minor effects may be justified by noting that the resultant of two mutually perpendicular forces, one of which is 10% of the other, is less than 1% greater than its largest component. (In some cases, deflection readings of the pier caps, Table 3, could not be taken because either the piles or the loading devices interfered with the placing of the gages.)

TABLE 3.—DEFLECTION OF PIER CAPS, IN THOUSANDTHS OF AN INCH (Positive Unless Otherwise Indicated)

Point	PART I.—NORTH-SOUTH ICE				PART II.—EAST-WEST ICE			
	Station 0		Station 1		Station 0		Station 1	
	North-south	East-west	North-south	East-west	East-west	North-south	East-west	North-south
1	34.0	2.5	36.7	0.0	....	....	82.5	−2.3
2	34.5	−0.8	36.4	−0.2	....	....	79.0	1.0
3	35.0	−1.8	36.9	−0.4	56.7	−1.1	80.1	0.5
4	35.2	1.3	37.1	2.4	57.8	−1.7	81.0	−2.0

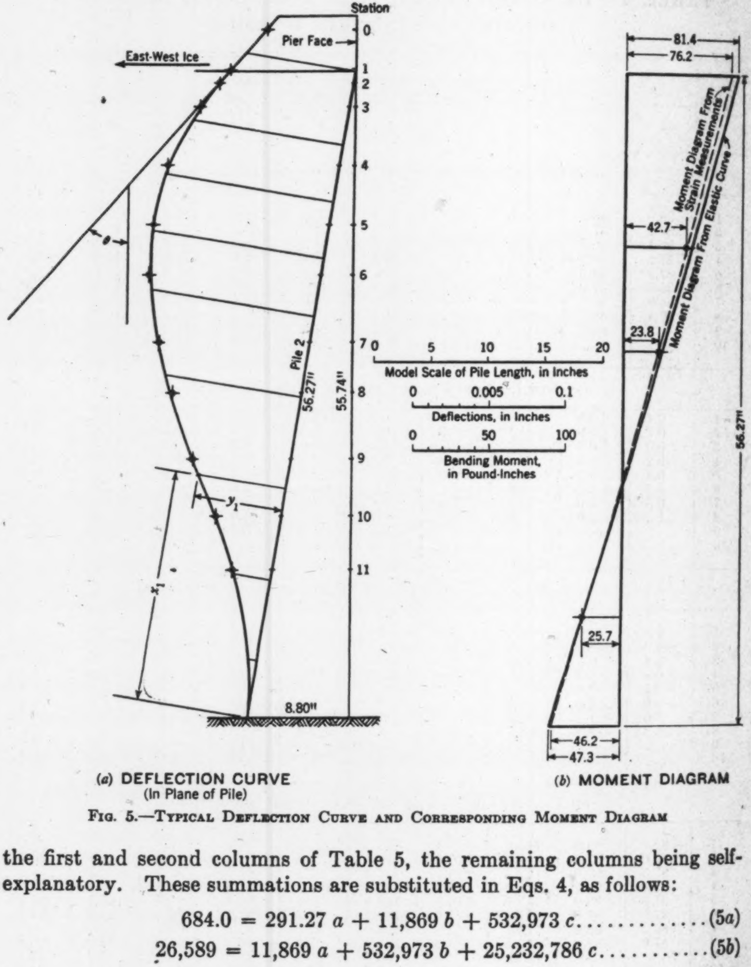
The equation of the elastic curve of pile 2 for the east-west ice load will be developed for purposes of illustration. The deflection curve is plotted in the plane of the pile's deflection as shown in Fig. 5. Each battered pile appears as such, its indicated length is its true length, and the deflections are plotted, as

TABLE 4.—DEFLECTIONS OF PILES, IN THOUSANDTHS OF AN INCH (Positive Unless Otherwise Indicated)

Pile	DEFLECTION AT STATION (SEE TABLE 2):									
	2	3	4	5	6	7	8	9	10	11
(a) PART I.—NORTH-SOUTH ICE; NORTH-SOUTH DEFLECTIONS										
1	38.0	38.5	37.4	34.2	30.6	23.9	18.6	12.7	7.6	4.0
2	37.2	37.6	36.6	34.0	30.8	25.7	20.9	15.5	10.1	6.2
3	36.1	36.3	35.9	33.0	29.1	24.9	19.8	14.1	9.1	5.5
4	37.2	38.2	37.0	35.1	32.6	27.7	22.8	17.1	11.6	7.1
5	37.2	38.0	37.0	35.7	32.7	28.2	24.0	17.8	12.9	7.9
6	36.5	37.0	34.9	32.2	28.7	24.0	19.2	13.4	8.6	5.1
7	36.9	37.7	35.7	32.5	28.8	23.2	18.4	13.3	9.2	5.8
8	37.4	38.1	38.0	35.0	31.8	26.2	21.5	15.0	10.0	5.7
9	....	....	....	34.0	30.7	25.0	20.3	14.0	9.3	5.7
10	....	....	....	33.8	30.0	24.3	19.0	14.0	9.2	5.8
(b) PART I.—NORTH-SOUTH ICE; EAST-WEST DEFLECTIONS										
1	0.4	0.6	0.7	−1.0	−0.3	−1.5	−1.9	−1.9	−1.1	−1.1
2	0.7	0.9	−0.1	−0.9	0.0	0.0	−0.3	−0.3	0.0	−0.4
3	0.3	0.3	−0.2	−2.7	−2.1	−4.0	−4.0	−4.1	−3.0	....
4	0.2	0.8	0.2	−0.3	0.9	0.5	0.5	0.2	0.7	0.5
5	0.4	0.0	−0.9	0.0	−0.1	−0.5	−0.5	−0.9	−1.0	−0.7
6	1.5	1.2	0.6	−0.5	−1.8	−2.6	−3.0	−2.6	−2.0	−1.3
7	0.3	2.0	2.1	2.0	0.6	1.8	1.0	0.2	0.0	....
8	1.2	2.6	1.7	1.6	1.8	2.2	1.8	1.1	1.2	1.0
9	....	....	....	....	0.6	1.0	0.0	0.0	0.2	0.0
10	....	....	....	−0.3	−0.3	0.0	0.0	−0.2	0.0	0.2
(c) PART II.—EAST-WEST ICE; NORTH-SOUTH DEFLECTIONS										
1	0.0	0.9	−1.4	1.0	−2.4	−0.6	−1.1	−1.0	0.0	−0.1
2	−1.9	−3.0	−1.9	−0.7	−3.6	−2.4	−1.8	−1.6	−1.0	−0.9
3	0.8	0.3	0.0	4.4	1.4	0.0	0.0	1.1	1.2	0.0
4	0.8	−1.4	−2.1	0.0	−4.7	−3.5	−3.0	−2.4	−2.0	−1.9
5	−0.7	0.0	1.7	−2.1	−0.5	0.2	1.1	2.0	2.6	1.0
6	−2.0	−2.0	−4.6	−2.8	−0.7	−0.8	0.4	0.0	1.0	0.0
7	2.0	−1.8	−2.0	−2.0	0.0	−2.1	−1.6	−0.3	0.5	−0.3
8	−2.3	−4.9	−3.7	−5.1	−3.7	−2.6	−3.1	−0.7	−0.9	−1.0
9	....	....	....	....	1.0	0.1	1.2	0.8	0.7	0.4
10	....	....	....	0.0	0.3	−1.7	−0.6	0.0	0.0	−0.4
(d) PART II.—EAST-WEST ICE; EAST-WEST DEFLECTIONS										
1	88.7	99.0	115.8	121.1	118.3	105.1	90.7	67.8	47.0	29.0
2	87.0	97.1	112.4	116.0	112.5	98.5	84.3	63.9	43.1	26.1
3	87.3	98.0	112.3	115.1	111.2	97.3	81.9	60.0	41.7	....
4	87.6	98.2	115.0	120.4	116.9	104.1	87.8	64.9	44.3	27.8
5	87.3	98.0	113.5	116.9	113.0	99.4	84.4	62.2	43.1	26.9
6	87.9	97.8	111.1	114.0	110.1	95.8	81.1	59.5	41.9	24.9
7	86.2	95.9	110.1	112.8	108.0	95.3	81.6	60.1	42.2	....
8	87.6	97.8	112.8	115.3	111.4	97.2	83.2	62.0	43.9	26.8
9	....	....	....	114.4	106.6	93.6	79.4	58.2	39.7	23.7
10	....	....	....	115.1	110.5	97.0	81.9	60.4	42.0	25.0

measured, horizontally. The points representing the displacement of the pier cap are the deflections of the center of the south cap as determined by averaging the deflections of points 1 and 4 (Table 3).

In developing the equation of the deflected pile, convenient abscissas are located along the pile axis, and the corresponding ordinates, in thousandths of an inch, are measured perpendicular to that axis. These coordinates appear in the first and second columns of Table 5, the remaining columns being self-explanatory. These summations are substituted in Eqs. 4, as follows:


Station
0
1
2
3
4
5
6
7
8
9
10
11
Pier Face
East-West Ice
θ
Pile 2
56.27"
55.74"
y_1
x_1
8.80"
0 5 10 15 20
Model Scale of Pile Length, in Inches
0 0.005 0.1
Deflections, in Inches
0 50 100
Bending Moment, in Pound-Inches
81.4
76.2
Moment Diagram From Strain Measurements
Moment Diagram From Elastic Curve
42.7
23.8
56.27"
25.7
46.2
47.3

(a) DEFLECTION CURVE (In Plane of Pile)

(b) MOMENT DIAGRAM

FIG. 5.—TYPICAL DEFLECTION CURVE AND CORRESPONDING MOMENT DIAGRAM

$$684.0 = 291.27\,a + 11{,}869\,b + 532{,}973\,c \qquad (5a)$$

$$26{,}589 = 11{,}869\,a + 532{,}973\,b + 25{,}232{,}786\,c \qquad (5b)$$

and

$$1{,}129{,}171 = 532{,}973\,a + 25{,}232{,}786\,b + 1{,}236{,}748{,}279\,c \qquad (5c)$$

Then, solving simultaneously, the following expression represents the elastic curve of pile 2 as deflected by the east-west ice load:

$$y = 0.3989\,x + 0.1737\,x^2 - 0.002803\,x^3 \qquad (6)$$

TABLE 5.—DEVELOPMENT OF THE EQUATION OF THE DEFLECTED PILE

y	x	x^2	x^3	x^4	x^5	$x\,y$	$x^2\,y$
0.0	0.00	0	0	0	0	0	0
5.6	5.00	25	125	625	3,125	28	140
25.0	12.00	144	1,728	20,736	248,832	300	3,600
55.2	20.00	400	8,000	160,000	3,200,000	1,104	22,080
86.4	28.00	784	21,952	614,656	17,210,368	2,419	67,738
107.0	35.00	1,225	42,875	1,500,625	52,521,875	3,745	131,075
115.0	40.00	1,600	64,000	2,560,000	102,400,000	4,600	184,000
113.5	45.00	2,025	91,125	4,100,625	184,528,125	5,108	229,838
101.3	50.00	2,500	125,000	6,250,000	312,500,000	5,065	253,250
75.0	56.27	3,166	178,168	10,025,519	564,135,954	4,220	237,450
684.0	291.27	11,869	532,973	25,232,786	1,236,748,279	26,580	1,129,171

The second derivative of this equation is

$$\frac{d^2y}{dx^2} = 0.3474 - 0.016818\,x \ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

and, if this is substituted in Eq. 1, the moment can be determined at any point along the pile axis. Thus, since $M = \frac{30{,}000{,}000 \times 0.004525}{1{,}000}\frac{d^2y}{dx^2}$ $= 135.75\,(0.3474 - 0.016818\,x)$, the two end moments are as follows: For $x = 0$, $M = 135.75\,(0.3474) = 47.16$ lb-in., and, for $x = 56.27$, $M = 135.75\,(-0.5989) = -81.30$ lb-in. (In the foregoing solution, 1,000 is introduced in the denominator because, throughout this analysis, the deflections have been measured in thousandths of an inch.) The resultant shear is $S = \frac{47.16 + 81.30}{56.27} = 2.28$ lb, and the equivalent horizontal shear, obtained by a comparison of similar triangles (Fig. 2), is $S_h = \frac{2.28 \times 63.46}{64.33} = 2.25$ lb.

STRAINS

Strain measurements were taken at three principal levels located at 15 in., 24 in., and 47 in. below the base of the pier cap (see Table 2). The object of these measurements was to determine the axial load and bending moments in each pile, the former remaining constant throughout but the latter varying and, in view of the absence of lateral loads on the pile, having maximum values at its extremities. It was obviously impossible to measure the deformations at the end of each pile, but the gages were applied as close to the ends as physical obstructions and convenience of reading permitted. The middle point was located so as to lie above the point of contraflexure in most cases.

The algebraic sum of the measured strains at the ends of a diameter, multiplied by a suitable constant, will give the axial load in the pile, and the algebraic difference of these strains, multiplied by another constant, will give the moment in the pile, in the plane of the gages, at a point corresponding to the center of the gage length. Strain readings in a plane perpendicular to this will yield moments, which, when resolved with the first set, will give the maximum values. Only the moments in the plane of the pile's deflection were considered because, as in the case of the deflections, the moments in the plane perpendicular to this were sufficiently small to neglect without appreciably affecting the indicated maxima.

A typical set of strain readings is reproduced herein, as well as the method of computing the axial loads and the corresponding moments:

														Average strain
			0.434				0.430				0.438			−0.434
		0.410		0.457		0.407		0.453		0.431		0.445		
N-gage	0.880		0.470		0.927		0.520		0.973		0.542		0.987	
			0.738				0.735				0.735			−0.736
		0.718		0.758		0.712		0.757		0.730		0.740		
S-gage	1.205		0.487		1.245		0.533		1.290		0.560		1.300	

In this instance the gages were located in a north-south plane at station T, pile 1, and the structure was deflected by the north-south ice load. In each series of readings an average of the differences obtained by applying and removing the load was considered a single strain measurement. Three consistent readings (such as the foregoing) were considered satisfactory. The values of the constants previously mentioned can be determined by a consideration of the following expressions which represent the stress condition at opposite ends of a diameter:

$$S_n = \frac{P}{A} + \frac{M\,c}{I} \quad \text{(8a)}$$

and

$$S_s = \frac{P}{A} - \frac{M\,c}{I} \quad \text{(8b)}$$

If these equations are added:

$$\frac{P}{A} = \frac{S_n + S_s}{2} \quad \text{(9)}$$

But

$$S_n = E\,\delta_n = E \times \frac{\Delta_n}{8 \times 1{,}075} \quad \text{(10a)}$$

and

$$S_s = E\,\delta_s = E \times \frac{\Delta_s}{8 \times 1{,}075} \quad \text{(10b)}$$

in which Δ is the total deformation over an 8-in. gage length as recorded with a gage magnification of 1,075; and δ = unit deformation. Substituting Eqs. 10 in Eq. 9 and denoting $(\Delta_n + \Delta_s)$ by Δ':

$$P = \frac{A\,E}{8 \times 1{,}075} \times \left(\frac{\Delta_n + \Delta_s}{2}\right) = \frac{A\,E}{17{,}200} \times \Delta' = 119.4\,\Delta' \quad \text{(11)}$$

Similarly, subtracting Eq. 8*b* from Eq. 8*a*:

$$\frac{M\,c}{I} = \frac{S_n - S_s}{2} \quad \text{.......................}(12)$$

Substituting Eqs. 10 in Eq. 12 and denoting $(\Delta_n - \Delta_s)$ by Δ'':

$$M = \frac{I}{c} \times \frac{E}{8 \times 1{,}075} \times \left(\frac{\Delta_n - \Delta_s}{2}\right) = \frac{E\,I}{17{,}200\,c} \times \Delta'' = 20.87\,\Delta'' \quad ..(13)$$

Thus, in the case of the foregoing illustrative example, the axial load and the

TABLE 6.—LOADS AND MOMENTS DERIVED FROM MEASURED STRAINS

Pile No.	Part I.—North-South Ice										Part II.—East-West Ice		
	Axial Loads, in Pounds							Average Moments, in Pound-Inches at Stations:					
	Station R		Station S		Station T		Average load	R	S	T	R	S	T
	North-south	East-west	North-south	East-west	North-south	East-west							
1	140.0	143.0	141.2	141.6	139.7	141.3	−141.1	−7.2	−1.3	6.3	−46.5	−26.8	26.7
2	107.9	108.2	107.0	109.4	109.1	107.5	+108.2	−5.9	−2.3	6.2	−42.7	−23.8	25.7
3	167.9	168.5	166.4	167.2	168.8	170.4	−168.2	−5.5	−2.0	6.4	−43.3	−21.5	26.3
4	94.8	93.9	92.2	95.3	92.4	93.4	+ 93.7	−5.9	−3.4	6.8	−45.9	−26.6	27.9
5	132.9	132.5	137.2	135.6	134.5	137.1	+135.0	−5.5	−3.5	6.4	−43.1	−22.6	25.2
6	110.6	110.8	112.0	110.1	111.0	110.8	−110.9	−5.3	−1.8	6.1	−41.5	−22.1	24.9
7	164.5	162.1	163.6	163.8	163.2	162.6	+163.3	−4.3	−0.8	5.1	−39.8	−20.0	23.0
8	86.6	91.2	90.3	89.1	90.2	87.6	− 89.2	−7.0	−2.9	6.6	−42.2	−21.9	24.2
9	8.3	8.6	8.8	7.9	8.8	8.1	+ 8.4	−4.7	−1.7	5.5	−40.9	−19.5	25.0
10	7.3	8.4	6.0	6.2	9.3	5.7	− 7.1	−4.7	−1.4	5.4	−42.7	−21.5	26.7

TABLE 7.—AXIAL LOADS, IN POUNDS, DERIVED FROM MEASURED STRAINS; Part II.—East-West Ice

Pile No.	NORTH-SOUTH					EAST-WEST				Average of north-south and east-west averages	Percentage variation based on average
	Determinations				Average	Determinations			Average		
1	83.3 85.5 88.6	84.0 86.4 91.0	84.0 88.6	85.2 88.6	86.6	95.3 97.1 101.9	96.3 98.9	96.3 100.0	98.0	− 92.3	±6.2
2	129.2	131.3	132.6	....	131.0	130.4	131.3	133.0	131.6	−131.3	±0.2
3	77.4	79.8	83.8	....	80.3	80.5	81.7	....	81.1	− 80.7	±0.5
4	129.4 135.7	130.6	131.9	134.7	132.5	139.7 142.5	140.2 143.3	141.4	141.4	−137.0	±3.3
5	94.1	95.5	97.5	....	95.7	95.0 101.5	97.7 101.5	97.7	98.7	+ 97.2	±1.5
6	115.0	118.2	119.0	120.6	118.2	124.0	124.0	125.2	124.4	+121.3	±2.6
7	79.7 82.6	80.2	81.0	82.6	81.2	87.2 89.8	87.4 89.8	87.4	88.3	+ 84.8	±4.2
8	124.2 131.0	125.7 131.8	126.4	127.5	127.8	128.7 133.8	129.0	131.4	130.7	+129.3	±1.2
9	2.2 8.4	3.6 10 5	4.3	8.1	6.2	4.8	5.5	6.4	5.6	+ 5.9	....
10	+2.4 −0.5	+1.2 −1.9	+0.4 −2.4	0.0 −3.6	−0.6	−1.9 0.0	−1.4 0.0	−1.2 +1.2	−0.5	−0.5	....

bending moment are as follows: $P = 119.4\,[(-0.434) + (-0.736)] = -139.7$ lb; and $M = 20.87\,[(-0.434) - (-0.736)] = +6.30$ lb-in.

It would be impracticable to tabulate all of the recorded strains, but the resulting axial loads and bending moments, computed in the foregoing manner, are listed in Tables 6 and 7. The column headings for the axial loads indicate the plane of the gages during the various determinations, and the headings for the average moments refer to the points at which the pile moments in the planes of loading were measured. In compiling Table 7, relatively large discrepancies were observed between indicated axial loads in the two planes of measurement. A large number of readings taken at various points along the pile were necessary

TABLE 8.—Comparison of Moments, in Pound-Inches

Pile No.	Part I.—North-South Ice				Part II.—East-West Ice			
	Top Moments		Base Moments		Top Moments		Base Moments	
	From elastic curve	From strain measurements	From elastic curve	From strain measurements	From elastic curve	From strain measurements	From elastic curve	From strain measurements
1	−13.0	−12.7	12.5	11.9	−84.1	−81.2	47.5	48.0
2	− 9.6	−11.3	8.4	9.5	−81.4	−76.2	47.3	46.2
3	−11.1	−11.1	9.8	9.8	−78.0	−76.0	46.4	47.0
4	−10.8	−11.5	8.5	9.3	−84.7	−82.3	48.2	49.8
5	−10.4	−10.7	7.5	8.4	−67.8	−74.8	42.0	45.4
6	−11.5	−10.4	10.5	9.5	−63.8	−72.0	41.2	45.0
7	− 6.8	− 8.4	7.1	8.0	−63.9	−68.6	39.5	42.0
8	−13.2	−13.3	10.9	10.6	−63.6	−72.8	40.4	44.2
9	−11.5	− 9.5	10.2	8.5	−73.4	−71.5	48.8	46.0
10	−11.3	− 9.2	10.7	8.7	−77.0	−75.0	49.7	47.0
Average	−10.9	−10.8	9.6	9.4	−73.8	−75.0	45.1	46.1

TABLE 9.—Summation of Vertical and Horizontal Forces, in Pounds

Pile No.	Part I.—North-South Ice					Part II.—East-West Ice				
	Axial load	Vertical component	Horizontal Component		Horizontal shear due to bending	Axial load	Vertical component	Horizontal Component		Horizontal shear due to bending
			East-west	North-south				North-south	East-west	
1	−141.1	−139.2	−12.85	19.28	0.43	− 92.3	− 91.0	12.62	8.42	2.28
2	108.2	106.7	14.78	9.85	0.37	−131.3	−129.5	−11.97	17.95	2.19
3	−168.2	−165.9	−15.32	22.98	0.38	− 80.7	− 79.6	11.03	7.36	2.17
4	93.7	92.4	12.80	8.53	0.37	−137.0	−135.1	−12.49	18.73	2.36
5	135.0	133.2	−12.30	16.44	0.34	97.2	95.9	13.28	8.86	2.12
6	−110.9	−109.4	15.15	10.10	0.36	121.3	119.6	−11.06	16.58	2.09
7	163.3	161.1	−14.87	22.31	0.30	84.8	83.6	11.58	7.73	1.96
8	− 89.2	− 88.0	12.19	8.12	0.43	129.3	127.5	−11.79	17.67	2.09
9	8.4	8.4	0.0	0.0	0.32	5.9	5.9	0.0	0.0	2.12
10	− 7.1	− 7.1	0.0	0.0	0.32	− 0.5	− 0.5	0.0	0.0	2.20
Summation		− 7.8	− 0.42	119.61	3.62	....	− 3.2	+ 1.20	103.30	21.58
Percentage discrepancy		± 0.8	± 0.4	....	+2.7	....	± 3.7	± 1.3	....	+4.1

to obtain a fair average. The end moments obtained by extrapolation from these tabulated values are compared in Table 8 with those determined from the elastic curve.

TABLE 10.—SUMMATION OF MOMENTS (IN POUND-INCHES) ABOUT THE x-AXIS, IN THE $y\,z$ PLANES

Pile No.	PART I.—NORTH-SOUTH ICE					PART II.—EAST-WEST ICE		
	Moment at base of pile (1)	Vertical component of axial load (2)	Distance from x-axis, in inches (3)	Col. 2 times Col. 3 (4)	Total moment; Col. 1 plus Col. 4 (5)	Vertical component of axial load (6)	Distance from x-axis, in inches (7)	Moment; Col. 6 times Col. 7 (8)
1	12	139.2	−15.30	2,130	2,142	− 91.0	−15.30	1,392
2	9	106.7	− 0.63	− 67	− 58	−129.5	− 0.63	82
3	10	−165.9	− 2.30	382	392	− 79.6	− 2.30	183
4	9	92.4	12.37	1,143	1,152	−135.1	12.37	1,671
5	8	133.2	15.30	2,036	2,044	95.9	15.30	1,467
6	9	−109.4	0.63	− 69	− 60	119.6	0.63	75
7	8	161.1	2.30	370	378	83.6	2.30	192
8	11	− 88.0	−12.37	1,089	1,100	127.5	−12.37	1,577
9	8	8.4	− 6.50	− 55	− 47	5.9	− 6.50	− 38
10	9	− 7.1	6.50	− 46	− 37	− 0.5	6.50	− 3
Sum	+93	....	....	+6,913	+7,006	....	....	+ 102
Moment on pier (= 120 lb × 56 in.)				....	−6,720	....	....	....
Percentage discrepancy $\left(=\frac{7{,}006 - 6{,}720}{6{,}720}\right)$				....	+4.3	....	....	±1.5

TABLE 11.—SUMMATION OF MOMENTS (IN POUND-INCHES) ABOUT THE y-AXIS, IN THE $x\,z$ PLANES

Pile No.	PART I.—NORTH-SOUTH ICE			PART II.—EAST-WEST ICE				
	Vertical component of axial load (1)	Distance from y-axis, in inches (2)	Moment; Col. 1 times Col. 2 (3)	Moment at base of pile (4)	Vertical component of axial load (5)	Distance from y-axis, in inches (6)	Col. 5 times Col. 6 (7)	Total moment; Col. 4 plus Col. 7 (8)
1	−139.2	−5.87	817	47	− 91.0	−5.87	534	581
2	106.7	−8.80	−939	46	−129.5	−8.80	1,140	1,186
3	−165.9	−5.87	974	46	− 79.6	−5.87	467	513
4	92.4	−8.80	−813	49	−135.1	−8.80	1,189	1,238
5	133.2	5.87	781	45	95.9	5.87	563	608
6	−109.4	8.80	−963	44	119.6	8.80	1,052	1,096
7	161.1	5.87	946	41	83.6	5.87	491	532
8	− 88.0	8.80	−774	44	127.5	8.80	1,122	1,166
9	8.4	0.0	0	45	5.9	0.0	0	45
10	− 7.1	0.0	0	46	− 0.5	0.0	0	46
Sum	....	....	+ 29	+453	....	....	+6,558	+7,011
Moment on pier			....	(= 120 lb × 56 in.)				−6,720
Percentage discrepancy			±0.4	$\left(=\frac{7{,}011 - 6{,}720}{6{,}720}\right)$				+4.3

Tables 9, 10, 11, and 12 are included as a general check on the validity of the experimental determinations of the axial loads and bending moments based on strain measurements. Table 9 represents the summation of the vertical and horizontal forces; that is, the three spatial components of the axial loads. For example, in the case of the north-south ice load, where the applied load is horizontal and in a north-south direction, the sum of the vertical components should be zero, the sum of the east-west horizontal components should be zero, and the sum of the north-south horizontal components, when combined with the horizontal shears due to bending, computed as shown in the text at Eqs. 6 and 7, should equal the applied load of 120 lb. In this case, the agreement of the test data with these requirements is indicated by the following: The discrepancy in the summation of the north-south horizontal forces equals $\frac{123.23 - 120}{120} = +2.7\%$.

TABLE 12.—SUMMATION OF MOMENTS (IN POUND-INCHES) ABOUT THE z-AXIS, IN THE $x\,y$ PLANE AT THE BASE

Pile No.	PART I.—NORTH-SOUTH ICE							PART II.—EAST-WEST ICE				
	Moment of North-South Components			Moment of East-West Components			To-tal; Col. 3 plus Col. 6	Moment of North-South Components		Moment of East-West Components		Total; Col. 9 plus Col. 11
	North-south com-ponent (1)	Dis-tance from z-axis, in inches (2)	Mo-ment; Col. 1 times Col. 2 (3)	East-west com-ponent (4)	Dis-tance from z-axis, in inches (5)	Mo-ment; Col. 4 times Col. 5 (6)	(7)	North-south com-ponent (8)	Mo-ment; Col. 8 times Col. 2 (9)	East-west com-ponent (10)	Mo-ment; Col. 10 times Col. 5 (11)	(12)
1	19.28	−5.87	−113	−12.85	−15.30	197	84	12.62	− 74	− 8.42	129	55
2	9.85	−8.80	− 87	14.78	− 0.63	− 9	− 96	−11.97	105	−17.95	11	116
3	22.98	−5.87	−135	−15.32	− 2.30	35	−100	11.03	− 65	− 7.36	17	− 48
4	8.53	−8.80	− 75	12.80	12.37	158	83	−12.49	110	−18.73	−232	−122
5	18.44	5.87	108	−12.30	15.30	−188	− 80	13.28	78	− 8.86	−136	− 58
6	10.10	8.80	89	15.15	0.63	10	99	−11.06	− 97	−16.58	− 10	−107
7	22.31	5.87	131	−14.87	2.30	− 34	97	11.58	68	− 7.73	− 18	50
8	8.12	8.80	72	12.19	−12.37	−151	− 79	−11.79	−104	−17.67	219	115
9	0.0	0.0	0	0.0	− 6.50	0	0	0.0	0	0.0	0	0
10	0.0	0.0	0	0.0	6.50	0	0	0.0	0	0.0	0	0
Sum	....	....	....	....	....	....	+ 8	....	....	....	....	+ 1
Percentage discrepancy				....	....	....	+1.1	....	....	....	....	±0.1

The discrepancies in the other two directions are based on the variation from the average of the plus and minus totals, as shown in the following computations:

Discrepancy in summation of

$$\text{Vertical components} = \frac{\pm 3.9}{505.7} = \pm 0.8\%$$

$$\text{East-west horizontal components} = \frac{\pm 0.21}{55.13} = \pm 0.4\%$$

Tables 10, 11, and 12 indicate the summation of moments, of the axial load components and the base moments, about the x, y, and z axes, respectively. The base moments listed in Table 8 are in the planes of deflection of the various piles; and, for use in Tables 9, 10, 11 and 12, they must be corrected, by ratios indicated on Fig. 2, to give the corresponding moments in the vertical planes of summation. The discrepancies in the moment summations are computed in a manner similar to those in the load summations. Referring to Tables 10, 11, and 12, it is to be noted that all piles are considered cut at the pier base and the forces and moments act on the structure as shown in Fig. 6. Clockwise moments are assumed positive. Moments applied at the base of each pile are neglected except when adding the moments in the plane of loading (see Tables 11 and 12, Part I).

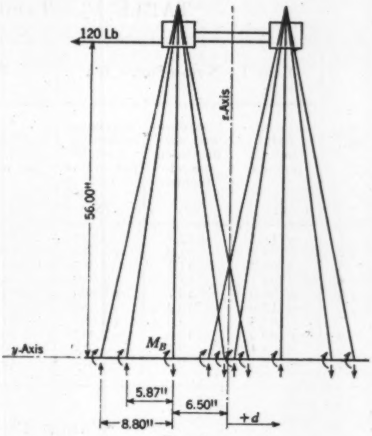


FIG. 6

DISCUSSION OF DATA

The major deflections for each loading vary consistently and give smooth deflection curves as indicated by the typical curve in Fig. 5. When the pier is subjected to a lateral force, especially the east-west ice load, the deflections at right angles to the plane of loading vary with respect to magnitude and sign but are sufficiently small to neglect when investigating the behavior of the pier. The moments corresponding to these secondary deflections are extremely small and have not been included in any summations or comparisons. Tables 9, 10, 11, and 12 show that the agreement between the measured and applied axial loads and moments is satisfactory, the maximum error being approximately 4%.

However, in the case of the direct load, the test data for which have been omitted, the sum of the vertical components of the axial loads is only 569 lb, or approximately 5% less than the 600-lb applied load. This discrepancy might be due to errors in the load-applying device, the measuring gages, or possibly to some effect peculiar to this loading. Since the axial loads were of reasonable magnitude, and sufficient readings were taken to obtain a fair average, and since the scales in the loading device were checked carefully, this discrepancy might be due to the method of applying the direct load.

SUMMARY OF TEST RESULTS

In Table 13, the axial loads and maximum end moments in the model have been summarized for the five conditions tested. The maximum stresses have been computed from these values, using the familiar expression for combined direct and bending stress, $\frac{P}{A} \pm \frac{M\,c}{I}$.

Each of the axial loads listed in Table 13 is the average of six determinations (Tables 6 and 7), and each of these determinations is the result of at least three consistent strain measurements as explained under the heading "Strains." The end moments in Table 13 were obtained from the moment curves established by strain measurements, these values being checked (see Table 8) by the moments based on the analytical treatment of the elastic curves as shown previously in the text. The end moments induced by direct loading (see Table 13, Part III) are negligible and were not included in the maximum stress computations.

TABLE 13.—TABULATION OF STRESSES

Pile No.	PART I.—NORTH-SOUTH ICE			PART II.—WIND LOAD			PART III.—DIRECT LOAD	
	Axial load, in pounds (1)	Maximum end moments, in pound-inches (2)	Maximum stress, in pounds per square inch (3)	Axial load, in pounds (4)	Maximum end moments, in pound-inches (5)	Maximum stress, in pounds per square inch (6)	Axial load, in pounds (7)	Maximum stress, in pounds per square inch (8)
1	−141.1	±12.7	−3,123	−88.8	±5.1	−1,723	−55.2	−806
2	108.2	11.3	+2,525	17.3	4.8	+ 654	−56.7	−857
3	−168.2	11.1	−3.385	−56.4	2.3	−1,016	−63.5	−927
4	93.7	11.5	+2,329	75.3	2.8	+1,334	−54.3	−793
5	135.0	10.7	+2,866	104.3	2.8	+1,757	−52.7	−770
6	−110.9	10.4	−2,489	−27.6	2.4	− 605	−57.0	−832
7	163.3	8.4	+3,087	37.9	4.2	+ 905	−56.8	−830
8	− 89.2	13.3	−2,415	−70.1	5.8	−1,509	−58.0	−847
9	8.4	9.5	+ 917	−20.4	4.5	− 674	−59.1	−863
10	− 7.1	9.2	− 873	19.6	2.4	+ 487	−60.1	−878

TABLE 13.—(*Continued*)

Pile No.	PART IV.—COMBINATION			PART V.—EAST-WEST ICE		
	Axial load, in pounds (9)	Maximum end moments, in pound-inches (10)	Maximum stress, in pounds per square inch (11)	Axial load, in pounds (12)	Maximum end moments, in pound-inches (13)	Maximum stress, in pounds per square inch (14)
1	−300.0	±19.4	−6,003	− 92.3	±81.2	−8,136
2	72.1	16.6	+2,441	−131.3	76.2	−8,288
3	−298.6	12.0	−5,364	− 80.7	76.0	−7,532
4	114.5	12.5	+2,717	−137.0	82.3	−8,881
5	175.3	11.8	+3,546	97.2	74.8	+7,673
6	−198.1	11.6	−3,863	121.3	72.0	+7,791
7	144.5	13.0	+3,197	84.8	68.6	+6,972
8	−230.0	19.6	−4,997	129.3	72.8	+7,973
9	− 79.8	18.7	−2,728	5.9	71.5	+6,063
10	− 45.5	12.0	−1,668	− 0.5	75.0	−6,276

The moments and deflections as determined experimentally for the combination loading (Part IV, Table 13) do not differ appreciably from the sum of the individual effects similarly determined, probably because of the batter of the piles. It may be concluded that the secondary effects are negligible and, therefore, need not be considered in the analysis or design of the prototype.

By use of the conversion factors given in the text following Table 1, the maximum stress determined in the prototype is 28,158 lb per sq in. This stress occurs in pile 1 when the pier is deflected by the least probable combination of loads (east-west ice, wind, and direct). The maximum lateral deflection determined in the prototype pier is 9.4 in. (east-west ice load) and the maximum longitudinal deflection is 6.2 in. (combination loading).

ACKNOWLEDGMENT

The complete record of data upon which this paper is based was submitted, by Mr. Coan, in 1938, as an essay to the School of Engineering, The Johns Hopkins University, Baltimore, Md., in conformity with the requirements for the degree of Master of Civil Engineering. A copy of the essay has been placed on file for reference in the Engineering Societies Library, 33 West 39th Street, New York, N. Y.

DISCUSSION

Jacob Feld,[5] M. Am. Soc. C. E. (by letter).—In a very clear manner the paper gives not only the report of tests on a model pier supported on battered column or pile supports, but also a method of designing such a structure. Of course, any design must be based upon the assumed action of the supporting piles or columns. Similarly, the construction of the test model was based on an assumed action of the supporting tubes which are called "piles."

There is a question whether the model should not be classed more properly as a pier supported on clusters of battered columns, corresponding to a possible construction for a very high viaduct. The paper states that in the prototype piles 200 ft long were used, but these piles were 30-in. shells, $\frac{7}{16}$ in. thick. To drive piles of that type for a depth of 200 ft, it is almost a self-evident conclusion that the material encountered must be very soft. However, if the material has no resistance, and therefore provides no skin friction, the question can be raised whether each of the piles or tubes can be considered safe as an individual column 30 in. in diameter and 200 ft long between bracing levels.

If the material is as soft as is stated, then considerable difficulty in sinking caissons to such depths is eliminated, and one is led to question whether a caisson design might not have been more economical. Perhaps the authors, in their concluding discussion, can advise whether a comparative study was made between a caisson design and the type shown.

By using a model ratio of $\frac{1}{40}$, a test specimen which was still of considerable height was used. However, although there can be no question as to the qualitative results, the final conclusion that under the assumed loadings a maximum lateral deflection of 9.4 in. and a maximum longitudinal deflection of 6.2 in. will result in the actual structure must be taken with some question.

In the actual completed structure, it would have been possible to measure the deflection of the pier upon imposing known horizontal pulls, not necessarily as large as the maximum expected and, in this way, provide a complete check on test results.

As it is also concluded by the authors, from the test results, that the moments and the deflections for combination loading do not differ appreciably from the sum of the individual effects similarly determined, the structure is apparently acting as an elastic body. Therefore, the determination of the deflection for smaller loads than the maximum expected can be used safely as a guide for the determination of maximum deflections under maximum loads.

The most serious objection to the model tests is the assumption of no restraint of the material through which the piles were driven. As stated previously, such an assumption is equivalent to a statement that these piles are columns which had a ratio of diameter to length of 80—considerably more than any unbraced columns normally used in construction design. Under the action of horizontal forces, an unrestrained pile would act as a column very closely approximated by a column rigidly held at the top and pin-connected at the bottom. In the model this condition was approximated closely, and the deflection curves are consistent with such an assumption.

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[5] Cons. Engr., New York, N. Y.

For a pile of this length, with restraint due to skin friction and reaction of the soil in which the lower ends are embedded, the deflection curve would be somewhat different, the point of contraflexure would be higher, and the affected unsupported height would be considerably less. It is safe to assume that the resulting stresses would be lower than those indicated by the tests. However, the test cannot be considered as a complete solution of the action of piles arranged as shown in Fig. 1 when driven into a resisting medium.

In the usual case of piles driven into soils, each cluster would act as a portal under either lateral or longitudinal loads, and there would be a redistribution of the resultant moments and deflections to equalize, more closely, the reactions of the individual piles in each cluster.

Although, as shown by the authors, the assumptions made in the first method for computing moments and deflections from an elastic curve are not as accurate, as far as determining resulting reactions, as those in the second method, which actually provides for measuring strains on opposite sides of piles, the closeness of the results is valuable for decreasing the work in future tests of similar structures. There can be no question at all as to the accuracy of the results based upon the measurement of actual strains, as long as the structure remains within the elastic limit of the materials.

The procedure outlined in analyzing the test data can be used directly in designing similar structures for determining the resultant reactions, in each pile, from vertical as well as horizontal loads.

In the closing discussion it is hoped that the authors will make available any records of pile-driving resistances from the actual construction, so that an appraisal can be made, even if only qualitatively, of the assumption that there is no skin friction along the face of the piles and no restraint to bending of the embedded portions.

THOMAS F. COMBER, JR.,[6] M. AM. SOC. C. E., AND JOHN M. COAN, JR.,[7] JUN. AM. SOC. C. E. (by letter).—The writers are indebted to Mr. Feld for his discussion and for his general approval of the paper. Mr. Feld questions the safety of such long piles in actual construction. The purpose of making these tests was to answer this question and to determine by actual measurements the stresses in, and the action of, such unprecedented piles. However, in the actual construction, the pier caps were extended to a level just below the mud line, and this considerably reduced the unsupported length and the deflections of the prototype pier.

Mr. Feld also suggests the possibility that, in such soft material, caisson construction might have been feasible. In this particular case, with more than 100 ft of soft mud in some places, the cost would have been out of the question if the piers were to be extended to solid material or rock.

The writers made clear, in the paper, their reasons for neglecting the restraining action of the soil. Mr. Feld agrees that this will give a more critical stress condition than actually would exist in the prototype.

In discussion, the writers had hoped to develop some mathematical analyses for such pile groups, using the test data as a check. This was the main purpose of submitting the results of this research to the profession, and no claims were made as to the practicability of the proposed construction.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 2119

THE GRAND CENTRAL TERMINAL IN PERSPECTIVE

By William J. Wilgus,[1] Hon. M. Am. Soc. C. E.

With Discussion by Messrs. F. Lavis, E. R. Hill, Alonzo J. Hammond, H. L. Ripley, A. J. Meehan, J. P. Hallihan, Arthur V. Sheridan, C. E. Smith, Harold M. Lewis, Bion J. Arnold, Jay Downer, and William J. Wilgus.

Synopsis

This paper is intended more as an exposition of the inception, development, and quickening of a complex organism of magnitude, dating from 1831, than as a detailed description of the manner in which each of its many parts has been designed and constructed. Within this framework appears an outline of the results that have been achieved in the interest of the owner, and of the public to whom they spell a successful adventure in civic planning—all made possible by the utilization of overhead air rights incident to the change of motive power from steam to electricity. Included also is an intimation of problems ahead, to be faced and solved.

To the engineer who reads the paper will perhaps come the reminder that men of his profession, endowed with vision, may evoke as well as direct "the great sources of power in Nature for the use and convenience of man," without necessarily the promise of self-enrichment customarily demanded by the business man of genius. To him, moreover, will come the thought that it is not only the railroad station he traverses, visible to the eye, that is affording him service and is entitled to his consideration, but also the vast unseen plant of which the station itself is the outward sign.

Introduction

Purpose to Be Served.—**The completion of engineering works of magnitude is usually marked by their description in the annals of the Society or in the technical press. The calling of the engineer is thereby enriched, and its claim to inclusion among the learned professions is increasingly strengthened. In one respect, however, it would seem that this laudable practice has fallen short**

Note.—Published in October, 1940, *Proceedings*.

[1] Weathersfield, Ascutney P. O., Vt.

of perfection. Seldom, if ever, do after years bring forth a subsequent statement of the extent to which the work in question may have measured up to expectations in whole or part. Members of the profession and the public at large too frequently are not apprized of its merits and faults, of which adequate knowledge is to be gained only through long-time experience. To be fairly judged and truthfully recorded on the pages of history, the work should be seen in perspective. It is to the interest of the owner, as well as of the engineering profession, that a deserved reputation for having well served a public purpose shall be made known, apart from the gaining of a monetary reward which also has proved in the end to have crowned the work with success.

It is in this spirit that the writer is venturing to lay before the profession the evolution of the Grand Central Terminal and its approaches in New York, N. Y., and vicinity, not alone because of the service that they have rendered to their owners and the travelers who use them, but also because of their contribution to enlightened civic planning.

EARLY BACKGROUND FROM 1831 TO 1869

Beginnings of the New York and Harlem Railroad.—The marvelous growth of the City of New York, confined within the limits of the island of Manhattan until 1874, and expanded to include the other four boroughs (Brooklyn, Queens, Bronx, and Richmond) after 1898, is matched by the striking advances that have been made in its means of trunk-line transportation since they had their start as a horsecar railroad more than a century ago.

In 1831, shortly after steam railroading devoted to public service first came to life in the United States, the New York and Harlem Railroad was incorporated; and its initial stretch of a little less than a mile was put into use with horses as the motive power between Prince and 14th streets in the following year. At that time the city's population was not much in excess of 200,000, and its projected street system north of 34th Street was as yet unopened and ungraded. Six and a half miles away was the isolated little village of Harlem, and between it and the city was the hamlet of Yorkville. The rugged mid-island terrain that faced the infant enterprise was indeed formidable for the times. Five years were required for the road to reach Harlem, in 1837, and another five to bridge the Harlem River and arrive at the point north of Williams Bridge, in 1842, where later the New York and New Haven Railroad, in 1849, was to make its connection at Woodlawn Junction, N. Y. Meantime, in 1839, the road had been extended southerly from Prince Street for a little more than a mile to the City Hall, and still farther in 1851 to the Astor House. Twenty years had elapsed since the work was started. In another year (1852) the road had completed its way to a junction with the Boston and Albany Railroad at Chatham, 130.75 miles from its southern terminus in New York.

In the initial distance of nearly 16 miles, double-tracked by this time as far as Woodlawn Junction, the larger portion of the line south of the Harlem River had its place in city streets, either from the beginning, as for example in the Bowery, or later in Fourth Avenue and its continuation (subsequently to be known as Park Avenue) north of 32d Street after the company's independent 100-ft right of way by agreement with the city in 1837 had been incorporated within that thoroughfare, and then widened north of 34th Street to 140 ft.

The motive power of the road was limited by law to the use of horses south of 14th Street. North of that point, as a result of public pressure, the steam locomotive was constantly on the retreat, first to an engine terminal at 32d Street in 1846 and then to a location just north of 42d Street in 1859, from whence, a half a century later, it was to take its flight to far-off South Croton-on-Hudson (Harmon), N. Y., and North White Plains, N. Y. So for nearly forty years, from 1832 to 1871, railroad cars in Manhattan were individually drawn by horses for increasing distances as the use of the steam locomotive became more and more obnoxious to a population that, by 1870, had grown to approximately a million people.

Entry of the New York and New Haven Railroad.—The New York and New Haven Railroad, a link in a series of railroads originating at Boston and other points in New England, secured trackage rights over the Harlem road south of Woodlawn Junction on March 17, 1848, and in the next year for the first time operated its trains in this manner for about 15 miles to Canal Street, where its terminus, adjoining a station of the Harlem road, remained until 1857. In that year the two roads established separately operated passenger stations side by side, with twelve tracks in all, on a block on the west side of Fourth Avenue between 26th and 27th streets, reaching back to Madison Avenue immediately north of Madison Square; no buildings had as yet been erected or streets opened above 42d Street. There the principal railroad terminus in mid-Manhattan remained for fourteen years until the Grand Central Depot took its place, and the use of horses for hauling trunk-line cars on the Harlem road south of 42d Street was thenceforth abolished. It was the focus of the rapidly growing trade that had been fostered and developed by the widening network of railways in New England, shortly to be expanded by extensions to Montreal and Quebec, Canada.

Arrival of the Hudson River Railroad.—It now remains to sketch the origin and development of Manhattan's third railroad entrance skirting its western shore (see Fig. 1). Incorporated in 1846 under the name of the Hudson River Railroad, and completed from Canal Street to Spuyten Duyvil, N. Y., in 1847, it was opened for its entire length, 144 miles, to East Albany, N. Y., in 1851, two years after the two mid-Manhattan roads had put the city in rail communication with the New England states. Beginning at Chambers Street, quite a distance west of the City Hall terminus of the New York and Harlem Railroad, it took its course at grade through Hudson, Canal, and West streets; then through 10th Avenue and private property to a station at 30th Street, to which point it was operated first by horses and eventually by means of a "dumb engine" preceded by a man on horseback bearing a red flag; and thence in 11th Avenue, ultimately to be known popularly as "Death Avenue," and beyond, the trains were hauled by steam locomotives of the usual type.

Shortly after the completion of the Hudson River Railroad under the direction of John B. Jervis,[3] Hon. M. Am. Soc. C. E., the series of lines that joined Albany, N. Y., with Buffalo, N. Y., were consolidated, in 1853, under the name of the New York Central Railroad, and by means of a ferry across

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[3] For memoir, see *Transactions*, Am. Soc. C. E., Vol. XI (1882), p. 109.

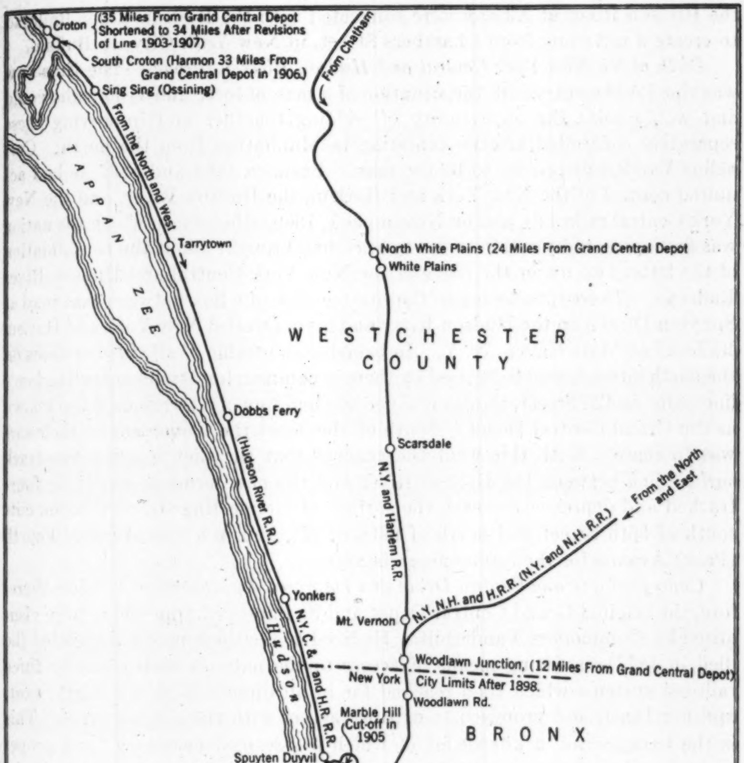
Croton
(35 Miles From Grand Central Depot Shortened to 34 Miles After Revisions of Line 1903-1907)
South Croton (Harmon 33 Miles From Grand Central Depot in 1906.)
Sing Sing (Ossining)
From the North and West
From Chatham
TAPPAN ZEE
Tarrytown
North White Plains (24 Miles From Grand Central Depot
White Plains
WESTCHESTER COUNTY
Dobbs Ferry
(Hudson River R.R.)
Scarsdale
N.Y. and Harlem R.R.
From the North and East
N.Y. N.H. and H.R.R. (N.Y. and N.H. R.R.)
Yonkers
N.Y.C. & and H.R. R.R.
Hudson
Mt. Vernon
Woodlawn Junction, (12 Miles From Grand Central Depot)
New York City Limits After 1898.
Woodlawn Rd.
Marble Hill Cut-off in 1905
BRONX
Spuyten Duyvil
S.D. and P.M. R.R. (N.Y.C. and H.R.R.R.)
Harlem River Drawbridge
Harlem (Hudson River R.R.)
River
Port Morris Branch
Port Morris
Mott Haven (5 Miles From Grand Central Depot)
N.Y. and Harlem R.R.
MANHATTAN
River
QUEENS
GRAND CENTRAL DEPOT (42nd ST.)
30th St.
Street Cars Only
East
St. Johns Park
Chambers St.
City Hall
Astor House
BROOKLYN
Upper Bay

FIG. 1.—RAILROAD ENTRANCES TO MANHATTAN, 1871-1874

the Hudson River at Albany were connected with the Hudson River Railroad to create a rail route from Chambers Street, in New York City, to Buffalo.

Birth of the New York Central and Hudson River Railroad.—The time now was ripe for the entry into the situation of a man of force and vision who could and would seize the opportunity of welding together and improving these separately controlled arteries centering in Manhattan from the north. Cornelius Vanderbilt proved to be the man. Between 1863 and 1867 he had acquired control of the New York and Harlem, the Hudson River, and the New York Central railroads, and on November 1, 1869—the year in which the nation was first spanned by rail from sea to sea—had brought about the consolidation of the latter two under the name of the New York Central and Hudson River Railroad. Thereupon he began the construction of a link between that road at Spuyten Duyvil on the Hudson River and his controlled New York and Harlem Railroad at Mott Haven, N. Y. In bringing the traffic of all three systems on the north into a funnel to be used by them in common for 5 miles of the Harlem's line as far as 42d Street, he also planned the building of a terminus, to be known as the Grand Central Depot. South of the depot the movement of their cars was to cease. With this went the concept that the then existing two-track surface line between the Harlem River and the new terminal should be four-tracked and depressed beneath the surface of intersecting streets in open cuts south of 56th Street and north of 96th Street, and in a tunnel under Fourth (Park) Avenue for the intervening distance.

Concept of a Grand Central Depot and Improved Approaches.—In 1869, therefore, the original Grand Central Depot and its improved approaches were visualized by Commodore Vanderbilt. He lived to see them partly completed (he died in 1877)—a fitting means of access to the nation's metropolis by three railroad systems which then reached far in all directions to the north, west, and northeast, and promised to expand farther with the coming years. This is the background, a knowledge of which has seemed necessary for a proper understanding of the course of events that have had their culmination in the Grand Central Terminal of the present day.

GRAND CENTRAL DEPOT FROM 1869 TO 1899

Planning and Building the Depot and Yard.—The decision having been made in 1869 that the new terminus should be located on the site of the then existing steam locomotive facilities between 42d and 45th streets (see Fig. 2), the purchase of the needed additional lands between Madison and Lexington avenues as far north as 48th Street was undertaken. By October 7, 1871, the work was well advanced, and the announcement was made that the trains of the Harlem road, one of which had departed from the depot on that day, would begin regularly to arrive and depart from it two days later, to be followed a week thereafter by those of the New Haven road, and still a week later by those of the New York Central. Apparently the link along the east bank of the Harlem River connecting the tracks of the former Hudson River Railroad at Spuyten Duyvil with those of the New York and Harlem at Mott Haven was then sufficiently ready for use, although according to some accounts it was not until the following year that it was fully completed.

In completing this plan and that which was soon to follow, a four-track open cut between retaining walls was created in Park Avenue from 56th Street to 49th Street, south of which the yard, embracing some 10 miles of track, fanned out on the surface for storing cars as far as Madison Avenue north of 45th Street and for serving the depot with its frontage of 240 ft on the north side of 42d Street and 695 ft on the east side of Vanderbilt Avenue. The work, spreading over two years, was under the direction of Commodore Vanderbilt and his son, William H. Vanderbilt, and was designed and its construction supervised by Isaac C. Buckhout,[3] M. Am. Soc. C. E.

Defects in Design.—The new depot when opened was hailed by the press of the day as deserving the thanks of the traveling community for such a liberal provision for their wants, and the arched roof of the train shed was praised as magnificent in its stately dimensions and its sincerity and dignity of design (see Fig. 3). In responsible quarters, however, it was criticized severely for its want of unity in its different parts, its unattractive architectural design, and its "unfortunate" exterior color treatment. In one important particular it was spoken of as "a great blunder." The yard tracks on the surface acted as a veritable "Chinese wall" to separate the city into two parts for fourteen blocks —nearly three quarters of a mile—between 42d Street and 56th Street and forced the discontinuance of a leading north and south thoroughfare, then known as Fourth Avenue, between 42d and 49th streets. In time, too, the rising $1\frac{1}{4}\%$ gradient south of 56th Street, and the absence of independent switching leads alongside of the four-track main entrance, were to limit gravely the capacity of the terminal, which in a few years was to be called upon to handle two to three times the 164 trains per day that greeted the opening in 1871. In effect, the entrance to the depot yard was throttled down from four main tracks to two, south of 56th Street.

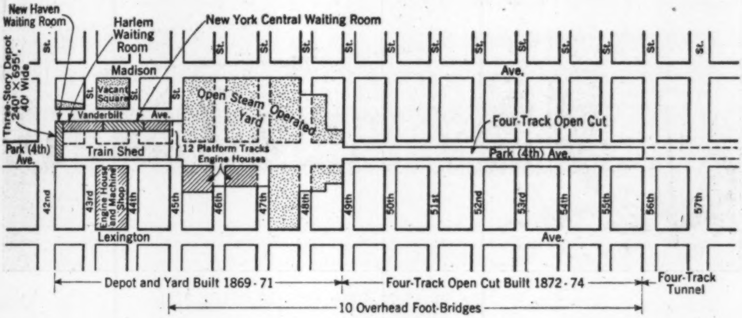
New Haven Waiting Room
Harlem Waiting Room
New York Central Waiting Room
Three-Story Depot 240' × 695', 40' Wide
St.
Madison
Ave.
Vacant Square
Vanderbilt
Ave.
Open Steam Operated Yard
Four-Track Open Cut
Park (4th) Ave.
Train Shed
12 Platform Tracks
Engine Houses
Park (4th) Ave.
42nd
43rd
Engine House and Machine Shop
44th
45th
46th
47th
48th
49th
50th
51st
52nd
53rd
54th
55th
56th
57th
Lexington
Ave.
Depot and Yard Built 1869-71
Four-Track Open Cut Built 1872-74
Four-Track Tunnel
10 Overhead Foot-Bridges

FIG. 2.—MAP OF DEPOT FACILITIES, 1871–1874

Fourth Avenue Improvement.—The Grand Central Depot was no sooner completed than plans were laid for the four-tracking of its approach as far north as the Harlem River. This was known as the Fourth Avenue Improvement and involved the building of an open cut between retaining walls from 49th Street to 56th Street (see Fig. 2); thence a tunnel to 96th Street; thence an open cut and a stone-arch viaduct to 115th Street; and thence an open cut to 133d Street, a monumental work, the cost of which was divided between the railroad company and the city. Its execution was entrusted to four commissioners, Alfred W. Craven,[4] Past-President, Am. Soc. C. E., Allan Campbell,[5] Hon. M. Am. Soc. C. E., and Edward H. Tracy,[6] M. Am. Soc. C. E., with Mr. Buckhout acting also as the engineer in charge. These four, and others on the work, such as Fayette S. Curtis,[7] Past-President, Am. Soc. C. E., and S. LeF. Deyo,[8] M. Am. Soc. C. E., in later years became prominent in the Society's affairs.

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[3] For memoir, see *Transactions*, Am. Soc. C. E., Vol. I (1872), p. 171.

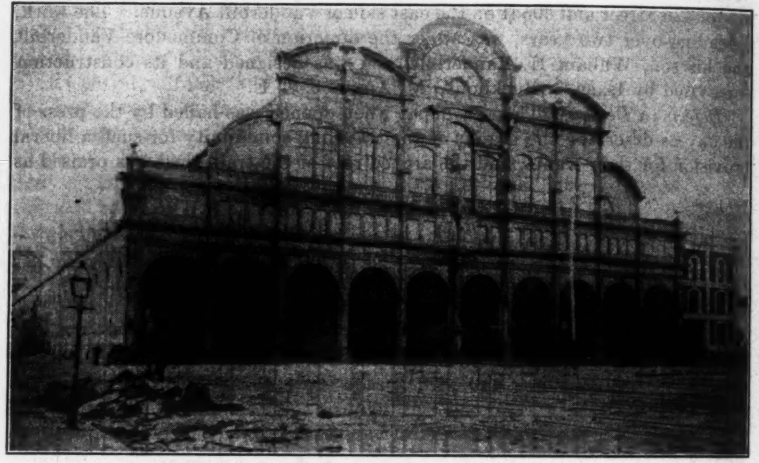

FIG. 3.—VIEW OF TRAIN SHED, LOOKING SOUTH, 1871

The construction of this adjunct to the usefulness and efficiency of the Grand Central Depot consumed about two years, from 1872 to 1874, during which, in 1872, the New York and New Haven Railroad became a part of the New York, New Haven and Hartford Railroad system; and, in 1873, the New York and Harlem Railroad was leased to the New York Central and Hudson River Railroad Company. Its outcome in a few years was to draw upon the owning company the opprobrium of the public at large and of passengers who were forced to travel through the covered section. The products of locomotive combustion in the two single-track side tunnels were imperfectly discharged by natural ventilation into the double-track center tunnel, from whence the smoke, gas, and cinders from all three tunnels were shot upward through longitudinal openings between the streets into the faces of passers-by and the occupants of the abutting property. The obscuration of signals in the tunnel and the distressing conditions to which the passengers were subjected in hot weather, together with injuries inflicted on the surroundings, all had their part in the initiation of the plan for a change of motive power from steam to electricity in 1899.

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[4] For memoir, see *Proceedings*, Am. Soc. C. E., Vol. VI (1880), p. 24.
[5] For memoir, *loc. cit.*, Vol. XX, October, 1894, p. 179.
[6] For memoir, see *Transactions*, Am. Soc. C. E., Vol. I, p. 377.
[7] For memoir, *loc. cit.*, Vol. 94 (1930), p. 1450.
[8] For memoir, *loc. cit.*, Vol. LXXXVI (1923), p. 1646.

Yard Improvements, Including the Annex.—With the passing of years other improvements were found to be necessary in order properly to meet a rapidly growing business, of which the first, in 1885–1886 (see Fig. 4), was the installation of improved interlocking switches and signals in an enlarged and rearranged yard. With this went the erection of a seven-track Annex, 100 ft wide and 650 ft long, on the easterly side of the depot for the benefit of incoming trains. The depot by this time contained twenty tracks of which eighteen were alongside of platforms. By an agreement dated December 31, 1885, horsecars were to be excluded from the depot confines, and a yard was to be provided at Mott Haven, five miles away, for the repair, cleaning, and storage of the New York Central's locomotives and through trains.

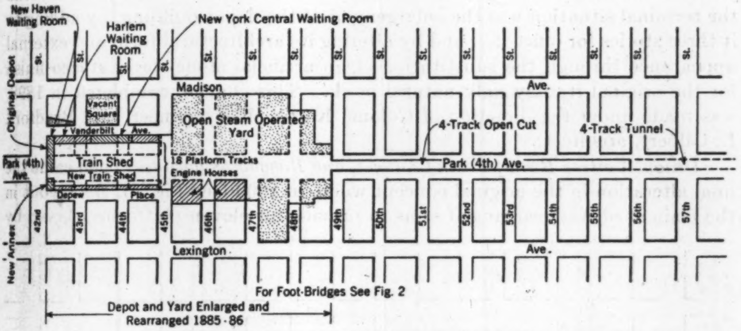
New Haven Waiting Room
Harlem Waiting Room
New York Central Waiting Room
Original Depot
St.
Madison
Ave.
Vacant Square
Vanderbilt Ave.
Open Steam Operated Yard
4-Track Open Cut
4-Track Tunnel
Park (4th) Ave.
Train Shed
New Train Shed
18 Platform Tracks
Engine Houses
Depew Place
42nd
43rd
44th
45th
46th
47th
48th
49th
50th
51st
52nd
53rd
54th
55th
56th
57th
New Annex
Lexington
For Foot-Bridges See Fig. 2
Depot and Yard Enlarged and Rearranged 1885-86

FIG. 4.—MAP OF DEPOT FACILITIES, 1885–1886

Harlem Depression.—The next change of magnitude was the depression and four-tracking of the Harlem road for 5 miles between a point just north of 138th Street and Woodlawn Road. This entailed the installation of automatic block signals, the elimination of grade crossings at twenty-two streets, and the building of five overhead passenger stations. The work lasted from 1888 to 1890 and was in charge of the chief engineer of the railroad, Col. Walter Katté,[9] M. Am. Soc. C. E. In 1891 improved signals were installed in the Park Avenue tunnel.

Park Avenue Viaduct and Harlem River Drawbridge.—The roadway, thus widened for four tracks at great expense, was not to be permitted to "rest in peace," however. Bending to popular clamor, the railroad officials, in the years 1893 and 1894, joined with the city in raising the tracks out of the depression in Park Avenue, between 106th Street and the Harlem River, and placing them on a four-track steel viaduct longer than 6,400 ft, north of 110th Street. Beyond the viaduct the railroad, at its own expense, substituted a high-level, four-track drawbridge, 398 ft long, with two approach spans on the north, for the old low-level bridge that had been a serious cause of train delays. The viaduct work, practically completed in March, 1895, was prosecuted under a "Board for the Park Avenue Improvement above 106th Street," for which Colonel Katté was the superintending engineer. In this he was aided by George H. Thomson,[10] M. Am. Soc. C. E., engineer of bridges, as he was in the case of the drawbridge and its approaches, completed in May, 1896, in which he acted as chief engineer for the railroad.

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[9] For memoir, see *Transactions*, Am. Soc. C. E., Vol. LXXXI, December, 1917, p. 1727.

Additional Stories and "Face Lifting."—The next move for improvement in the terminal situation was the enlargement of the depot building, by adding to it three stories for office use, and by altering its architectural form and external appearance through the substitution of an artificial stone-work, stucco finish for the original frankly ugly natural brick. This change, completed in 1898, was made under the direction of Colonel Katté, chief engineer, and Bradford L. Gilbert, architect.

Union Waiting Room, Track Changes, and Baggage Subway.—Then came the final alteration in the original concept when, in 1900 (see Fig. 5), the layout in the train shed was rearranged so as to provide for eleven platforms, served by nineteen tracks under cover, and a baggage subway at their north end, coupled with the slight shortening of the stub tracks at their south end in order that a 90-ft by 180-ft single waiting room, 36 ft high, and a 30-ft concourse might be provided between them and 42d Street, in place of the individual waiting rooms that had been used separately by the three companies during the previous twenty-nine years. Extensive changes, too, were made in the yard, including improvements in the signals and interlocking. The writer, as chief engineer, and Samuel Huckle, Jr., as architect, were responsible for the planning and direction of the station changes which, by the way, had the endorsement of Mr. Curtis, then chief engineer of the New Haven company. At last the depot (Fig. 6) with its yard (Fig. 7) was to be used in common by the owner and its two tenants as a union station in the full meaning of the term. Its name had become the Grand Central Station and it looked as if, with its approaches from the north, it had reached a stage of perfection, barring the dreadful smoke con-

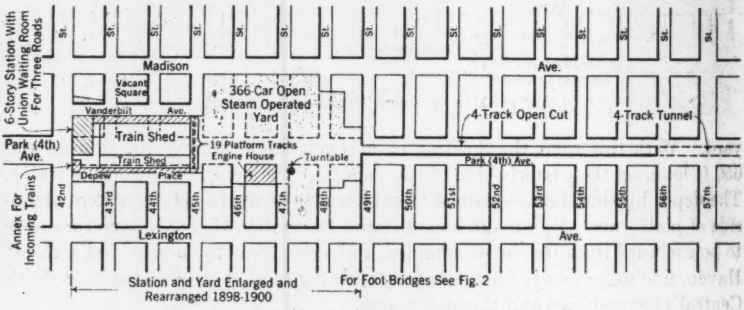
6-Story Station With Union Waiting Room For Three Roads
Park (4th) Ave.
Annex For Incoming Trains
St.
Madison
Ave.
Vacant Square
Vanderbilt
Ave.
Train Shed
Train Shed
Depew
Place
366 Car Open Steam Operated Yard
19 Platform Tracks
Engine House
Turntable
4-Track Open Cut
4-Track Tunnel
Park (4th) Ave.
42nd
43rd
44th
45th
46th
47th
48th
49th
50th
51st
52nd
53rd
54th
55th
56th
57th
Lexington
Ave.
Station and Yard Enlarged and Rearranged 1898-1900
For Foot-Bridges See Fig. 2

FIG. 5.—MAP OF DEPOT FACILITIES, 1898–1900

ditions in the yard and Park Avenue tunnel, the severance of the city's street system in a region of rapidly growing importance, and the lack of switching leads at the throat of the yard. Certainly there was nowhere in the United States, if in the world, a more noteworthy railroad terminal, with its enormously costly 12-mile four-track entrance, without grade crossings, from New England and Canada, connecting 5 miles out with a double-track line bearing traffic

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[10] For memoir, *Transactions*, Am. Soc. C. E., Vol. LXXI, March, 1911, p. 438.

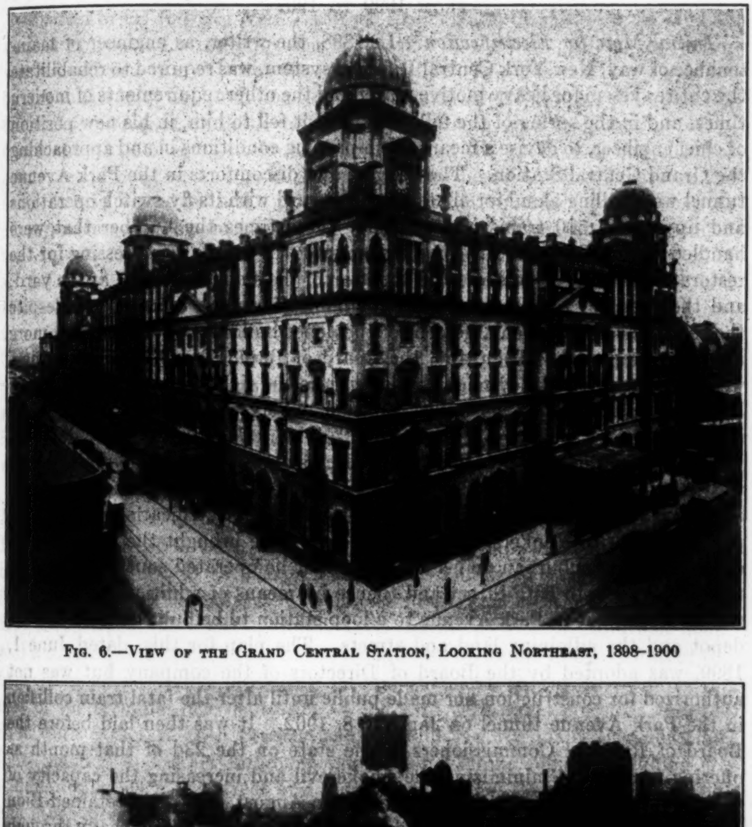

FIG. 6.—VIEW OF THE GRAND CENTRAL STATION, LOOKING NORTHEAST, 1898–1900

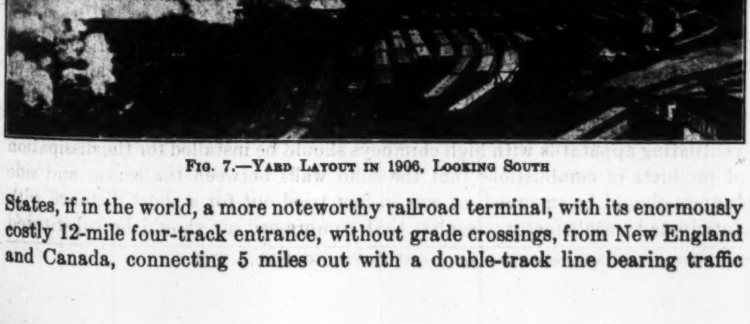

FIG. 7.—YARD LAYOUT IN 1906, LOOKING SOUTH

from the north and west as far as the St. Lawrence River and the Great Lakes and beyond to sub-arctic regions and the Pacific Coast.

GRAND CENTRAL TERMINAL TRANSFORMATION—FORMATIVE PERIOD FROM 1899 TO 1907

Initial Move for Electrification.—In 1898, the writer, as engineer of maintenance of way, New York Central Railroad system, was required to rehabilitate the entire system for heavy motive power and the other requirements of modern times, and in the spring of the following year it fell to him, in his new position of chief engineer, to devise a means for improving conditions in and approaching the Grand Central Station. The dangers and discomforts in the Park Avenue tunnel were calling aloud for abatement; the yard with its fly-switch operations and upward of 500 trains a day, nearly three times the number that were handled in the 1870's, had become inadequate; the public was pressing for the restoration of cross streets and the abolition of the smoke nuisance at the yard; and the needs of the three railroad companies for more office space, despite the recently completed enlargement of the depot building, were becoming more and more apparent. Then, too, the Board of Rapid Transit Commissioners were intimating that they might by law secure the right to route their subway beneath the terminal, to the exclusion of such underground usage as the railroad might find to be necessary in the future.

The writer's modest experience with electrical problems, gained in the mid-west, was fanned anew into flame by a visit from Frank J. Sprague,[11] M. Am. Soc. C. E., who called on the writer to propose the electrification of the company's branch to Yonkers, N. Y. This conference brought the writer to the conclusion that suburban trains at least might be operated southerly through the side tunnels to 56th Street and thence by means of additional tracks in a widened open cut in Park Avenue to a loop station to be built beneath the old depot and the adjoining land and streets. The plan for this, dated June 1, 1899, was adopted by the Board of Directors of the company but was not authorized for construction nor made public until after the fatal train collision in the Park Avenue tunnel on January 8, 1902. It was then laid before the Board of Railroad Commissioners of the state on the 23d of that month as offering a means of minimizing the smoke evil and increasing the capacity of the terminal. Before this date the writer, in August, 1901, had retained Bion J. Arnold, M. Am. Soc. C. E., to study the feasibility of handling heavy through trains by electricity between Mott Haven and the terminal, and his reports in February, 1902, were in the affirmative.

Meanwhile, the press of the city, particularly in the summer of 1901, was excoriating the Board of Directors for having done nothing, and all kinds of queer plans were advanced in its columns, and even in technical quarters, such as the proposal that the 2-mile tunnel should be operated as a single block; that ventilating apparatus with high chimneys should be installed for the dissipation of products of combustion; that the solid walls between the center and side tunnels should be removed, as was in fact tried out for a short distance with anticipated unsatisfactory results; that compressed air should be substituted for steam in the movement of trains; that the tunnel should be converted into an open cut; and that electric illumination should be installed in the tunnel, long since frowned upon by the enginemen who by experience had found it to be wanting.

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[11] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 100 (1935), p. 1736.

Concept of an Entirely New Terminal Utilizing Air Rights.—As he studied various proposals for retaining the old station in the new project, the writer was not satisfied that this was the ideal solution. In particular it was evident to him that a lofty new building for badly needed additional railroad offices on the site of the Annex would be in unhappy contrast with the neighboring imitation stone depot of less stately height. Why not (he questioned himself) tear down the old building and train shed and in their place, and in the yard on the north, create a double-level, under-surface terminal on which to superimpose office quarters and revenue producing structures made possible by the intended use of electric motive power? With this arrangement would go the erection of an adjoining hotel on the Harlem road's vacant square on Vanderbilt and Madison avenues between 43d and 44th streets (see Fig. 8), and the incorporation of a realty company for acquiring land and operating the rentable facilities. The keynote in this plan was the utilization of air rights that hitherto were unenjoyable with steam locomotives requiring the open air, or great vaulting spaces, for the dissipation of their products of combustion. Thus from the air would be taken wealth with which to finance obligatory vast changes otherwise nonproductive. Obviously it was the thing to do.

Plan of Revenue Producing Terminal and Improved Approaches.—It was on March 19, 1903, that the writer, as vice-president of the New York Central Railroad system, laid before President William H. Newman the detail plans that were intended to give expression to the parent idea conveyed to him in the preceding December. The objects were the erection of a suitable fifty-seven-track, all-electric, double-level terminal with a suburban loop, and a future rapid-transit connection should one be found to be desirable; the utilization of air rights producing income sufficient to pay interest on the cost of the terminal, the depression of the yard with street crossings restored from 45th Street to 55th Street, inclusive, and the attendant electrification, four-tracking, and other improvements to far-reaching points, including the elimination of 44 grade-street and highway crossings; the erection of a hotel and other means of attracting travel and revenue; the creation of a new north and south elevated city artery circumscribing the station building, extending over 42d Street on the south and joined on the north to a broad "Court of Honor" or "Grand Central Park" (see Fig. 9) on a future second level over the intersecting cross streets between 45th and 48th streets; and ramps connecting car-floor-level train platforms with the concourses. For this plan, Reed and Stem of St. Paul, Minn., were the architects for the station building. The opinion was hazarded that, in addition to being financially self-supporting, this treatment of the problem would result in the trebling or quadrupling of the suburban service when electrified.

As of May 6, 1903, the estimated cost of the improvements, exclusive of the cost of additional lands and interest during construction, extending from 42d

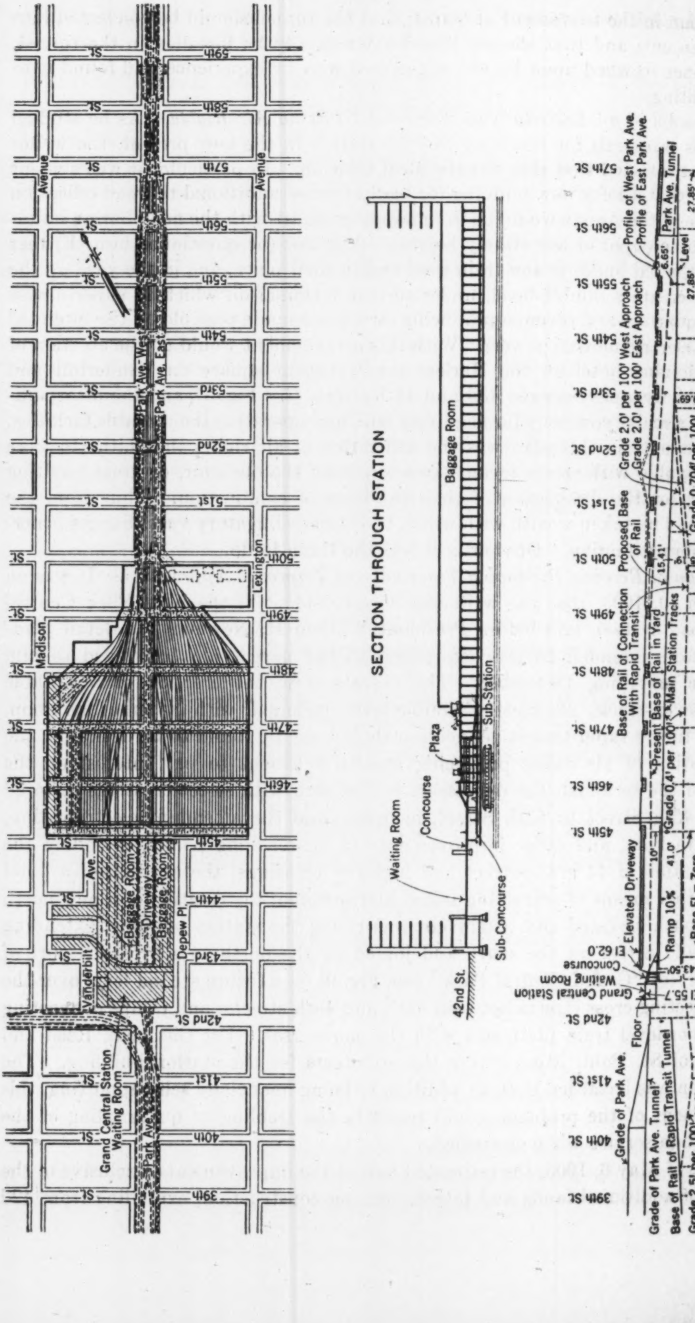
59th St.
58th St.
57th St.
56th St.
55th St.
54th St.
53rd St.
52nd St.
51st St.
50th St.
49th St.
48th St.
47th St.
46th St.
45th St.
44th St.
43rd St.
42nd St.
41st St.
40th St.
39th St.
Madison Avenue
Lexington Avenue
Park Ave. West
Park Ave. East
Vanderbilt Ave.
Depew Pl.
Baggage Room
Driveway
Baggage Room
Grand Central Station Waiting Room
Park Ave.
SECTION THROUGH STATION
Baggage Room
Waiting Room
Concourse
Plaza
Sub-Station
Sub-Concourse
42nd St.
Profile of West Park Ave.
Profile of East Park Ave.
Grade 2.0' per 100' West Approach
Grade 2.0' per 100' East Approach
Proposed Base of Rail
Base of Rail of Connection With Rapid Transit
Present Base of Rail in Yard
Main Station Tracks
Elevated Driveway
Ramp 10%
Grade 0.4' per 100'
Grade 0.704' per 100'
Base El - Zero
Concourse
Waiting Room
Grand Central Station
El 62.0
El 55.7
Floor
Grade of Park Ave.
Grade of Park Ave. Tunnel
Base of Rail of Rapid Transit Tunnel
Grade 2.5' per 100'
Park Ave. Tunnel
Level
27.85'
6.65'
29.66'
15.41'
10'
41.0'
43.50'
10.0'
61
33+60.83
28+39.17
20+56.67
8+00
1+75
0+00
-3+30

FIG. 8. PROPOSED TRANSFORMATION AS OF DECEMBER 22, 1902, PREDICATED ON THE UTILIZATION OF AIR RIGHTS

Street to Croton-on-Hudson and North White Plains, 35 and 24 miles, respectively, including a hotel on the Harlem company's vacant square, was as follows:

Station	$ 8,500,000
Yard	6,750,000
Electrification	10,400,000
Four-tracking, etc.	7,810,000
Revenue producing facilities	10,000,000
Total	$43,460,000

Therefore, 20% of the total was assignable to the station proper, 15% to the yard, 24% to the electrification, 18% to the four-tracking and kindred work within the electric zone, and 23% to the superimposed and adjoining revenue producing facilities, including the hotel. The annual net earnings to be realized from rentals and privileges alone were placed at $1,450,000, or enough to yield in excess of 3% on the entire outlay, apart from what was to be gained from economics in operation and from increased travel. In other words, enough revenue was believed to be obtainable from the voluntary investment of 23% of the total cost to carry in large part the 77% that was unavoidable in the interest of the railroad companies and the public.

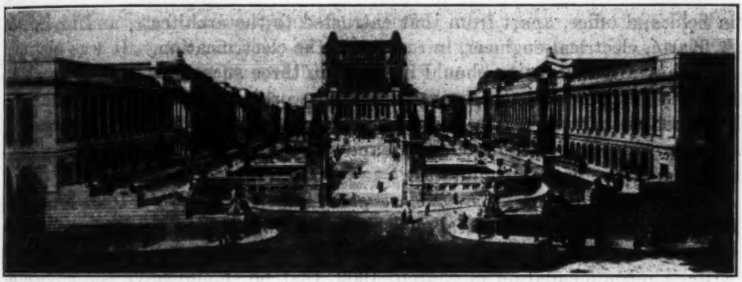

Fig. 9.—Proposed Transformation as of March 19, 1903, Looking South, Showing Park Avenue Restored and Widened to Create a "Court of Honor" Over the Cross Streets and Continued by Means of Circumscribing Elevated Driveways Around the Station and Over 42d Street

Approval of Plans by City and State and Commencement of Work.—On June 3, 1903, these plans, so much more comprehensive than those approved by the city in the preceding December, were laid before a special committee of its Board of Estimate and Apportionment, and were then declared by Mayor Seth Low to be proof of the thought he had had for some time that the changes which were to come about within a comparatively short time would entirely alter the complexion of the city. Nelson P. Lewis,[12] M. Am. Soc. C. E., chief engineer of the Board, said: "The plans impress me as providing perhaps the finest railway terminal station in the world. * * * I am deeply impressed with the magnanimous spirit of the company in planning things in a large and comprehensive way, without regard to cost."

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[12] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1413.

In their entirety, including the electrification south of the Harlem River, the plans were formally approved by the city on June 19, 1903, under the provisions of the Laws of 1903, Chapter 425, that had been adopted by the state and accepted by the city on May 7 of that year; and the date of commencement of the work was fixed at July 1, 1903, with the understanding that it was to be completed within five years thereafter. Actual construction was started on track changes at 47th Street on July 18 and in full earnest on August 17, when the contractor for the yard depression, the O'Rourke Engineering Construction Company (of which John F. O'Rourke,[13] M. Am. Soc. C. E., was President), began the demolition of buildings on Park and Lexington avenues between 45th and 50th streets. Olaf Hoff,[14] M. Am. Soc. C. E., was engineer of structures, later succeeded by J. L. Holst, bridge engineer. A. B. Corthell,[15] M. Am. Soc C. E., until well on in 1906, was resident engineer in charge of work in field and office, apart from that entrusted to the architects, and to Edwin B. Katté, electrical engineer, in respect to the electrification. It was planned that the yard excavation should be made in three successive "bites," each to be completed before another was undertaken, working westward from Lexington Avenue, so that the traffic of the three railroads using the terminal, by this time grown to more than 1,000 train and switching movements on a busy day, might continue without hindrance.

Concept of All-Electric Service Throughout Suburban Region.—It is now necessary to retrace steps for several months to the time when measures were undertaken to bring about the change in motive power. As an outcome of the writer's recommendation in March, 1902, that he should have the advice of experts in that respect, an Electric Traction Commission came into being on December 17 of that year, composed of Frank J. Sprague, Bion J. Arnold, and George Gibbs[16] (until September 5, 1905), Members, Am. Soc. C. E., as well as Arthur M. Waitt, the railroad's superintendent of motive power, shortly thereafter succeeded by J. F. Deems who took his place on the railroad. The writer made the fifth member and acted as chairman. Mr. Katté was made secretary and, as electrical engineer, was put in charge of the electrification work in field and office.

As expressed in his several reports to President Newman between 1901 and 1904, the writer's original concept was that the outward limits of the electric zone for both express and suburban service should be established at Croton-on-Hudson and North White Plains. This was despite the absence of a legal obligation to go farther north than the Harlem River, because it was his conviction that changing from steam to electric locomotives and vice versa within the city limits would seriously hamper traffic and would soon lead, by compulsion, to another banishment of steam still farther to the north. Equally strong was his belief that an all-electric frequent service in the suburban region would promote a marked increase in traffic, and a friendly attitude by the public, without which any corporation, railroad or otherwise, has "hard sledding." Nevertheless, there was a feeling by some in the Commission that electric locomotive service should not extend north of Mott Haven, because of the unpreparedness of the art for such a revolutionary change in the handling of extremely heavy, speedy, trunk-line trains. This was given voice in a letter dated June 8, 1903, to the writer from one of the Commissioners, who was supported in his position by another member. On October 31, 1903, a vote on the subject stood three to two in favor of the full extension, the writer casting the deciding vote. However, on the 3d day of the following month, the Commission unanimously decided that the plan as first promulgated should stand unchanged—namely that both multiple-unit suburban trains and express trains hauled by electric locomotives should be operated for the full distance between the Grand Central Terminal and the northerly termini at Croton-on-Hudson and North White Plains. Other important decisions reached at these and other meetings in 1903 were in favor of the use of direct current supplied to third-rail working conductors, and of vertical turbo-generators in the power stations. Dramatic conflicts of opinion developed between those weighted with the responsibility of making the enterprise work successfully and those on the outside who, without that responsibility, would see it done to their liking. Prominent among the critics was the late George Westinghouse, M. Am. Soc. C. E., who favored the adoption of alternating current supplied to overhead working conductors instead of direct current fed to a third rail, and who likewise favored the use of a horizontal turbo-generator. The A.C.–D.C. struggle later was to assume the dignity of a "celebrated case" comparable to the famous "battle of the gages" in England in bygone years. It gathered force when the New York, New Haven and Hartford Railroad, owning trackage rights south of Woodlawn, belatedly adopted the views of Mr. Westinghouse and found it necessary to plan an electric locomotive that could operate on both alternating and direct currents.

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[13] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 100 (1935), p. 1707.

[14] For memoir, *loc. cit.*, Vol. 89 (1926), p. 1623.

[15] For memoir, *loc. cit.*, Vol. 88 (1925), p. 1374.

[16] For memoir, *loc. cit.*, Vol. 105 (1940), p. 1840.

The deliberations of the Commission came to an end at their 108th meeting on October 5, 1906, after the initial electric zone had been completed and was ready for the running of the first electric train.

Intrusion of New Architects and Resultant Changes in Station Building Design.—It was not alone in the field of electrification that differences arose. After Reed and Stem had been accepted as the architects for the Grand Central Station building, the architectural firm of Warren and Wetmore in 1903 offered to the railroad company a plan of another type. Later the last named firm became a part of a new organization for designing the Grand Central buildings known as "Reed and Stem and Warren and Wetmore, Associated Architects," of which Charles A. Reed was named the executive, aided later by Mr. Corthell who had previously acted as resident engineer. The two new members of the firm did not look approvingly upon the revenue producing features of the original design and sought to change it to one more monumental in character, devoted solely to railroad purposes. In addition to disapproving the outside terraced driveways, they proposed that the treatment of the waiting room and concourse should be altered and the train room left unadorned. It must be confessed that the differing views in the end resulted in compromises that in some ways were praiseworthy, although the resulting abandonment of the elevated driveways around the station building and over the cross streets north and south of it (in which the writer and Vice-President E. H. McHenry, M. Am. Soc. C. E., of the New Haven road joined in non-concurrence) was a grievous mistake that remained unrectified in full for many years. Happily, the station was so built that the elevated driveways could be added in the future, if and when it was deemed to be desirable. The intended high office building over the south end of the station on 42d Street also "fell by the wayside"; but provision was eventually made for its future construction around the concourse on the line of 43d Street. In addition to this the revenue producing features "held their own" on the street and lower levels of the station and now remain to enrich the coffers of the railroad companies. This is true to a remarkable degree where the supporting columns beneath the street surface came to be designed for superimposed 20-story revenue producers outside of the 6-story station area.

Continuing Evolution of Plans.—During the years that elapsed from the date of the first agreement with the City of New York, on June 19, 1903, to the fourth one dated July 8, 1907, the plans for the station and yard and the approaches within the electric zone were in constant flux owing to changing conditions; the opening of the purse of the railroad company for the purchase of additional land; the further requirements of public authorities; and the advancement of new ideas. At the station, as has been mentioned, the circumscribing elevated driveways were eliminated and the intended skyscraper frontage on 42d Street abandoned in favor of a building of a uniformly moderate height covering the original site and also the adjoining area extending to Lexington Avenue between 43d and 45th streets. Likewise, the intended "Court of Honor" over the cross streets from 45th Street as far north as 48th Street, of a width equal to that of the main station building fronting on 45th Street, went into the discard (see Fig. 10). However, it was in part revived in after years when the restored circumscribing driveways were extended over 45th Street and thence on a descending grade to the present level of Park Avenue, of which the viaduct therein had been widened from 60 ft to 140 ft under the modified agreement with the city dated April 28, 1905.

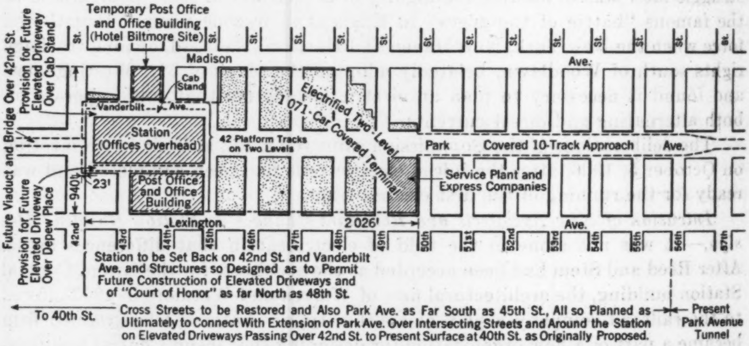
Temporary Post Office and Office Building (Hotel Biltmore Site)
Future Viaduct and Bridge Over 42nd St.
Provision for Future Elevated Driveway Over Cab Stand
Provision for Future Elevated Driveway Over Depew Place
Madison
Ave.
St.
Cab Stand
Vanderbilt Ave.
Station (Offices Overhead)
42 Platform Tracks on Two Levels
Electrified Two-Level 1 071-Car Covered Terminal
Park
Covered 10-Track Approach
Ave.
Service Plant and Express Companies
Post Office and Office Building
23'
940'
Lexington
2 026'
Ave.
42nd
43rd
44th
45th
46th
47th
48th
49th
50th
51st
52nd
53rd
54th
55th
56th
57th
Station to be Set Back on 42nd St. and Vanderbilt Ave. and Structures so Designed as to Permit Future Construction of Elevated Driveways and of "Court of Honor" as far North as 48th St.
To 40th St.
Cross Streets to be Restored and Also Park Ave. as Far South as 45th St., All so Planned as Ultimately to Connect With Extension of Park Ave. Over Intersecting Streets and Around the Station on Elevated Driveways Passing Over 42nd St. to Present Surface at 40th St. as Originally Proposed.
Present Park Avenue Tunnel

FIG. 10.—MAP OF PROPOSED TRANSFORMATION AS OF SEPTEMBER 30, 1907

Then, too, underground rights on that date were secured from the city in Vanderbilt Avenue and Depew Place, whereby the suburban station could be widened; the station front on 42d Street was set back 40 ft and on Vanderbilt Avenue 70 ft from the street lines; and the site of the 45th Street viaduct was moved southerly so as to be within the original lines of that thoroughfare. Rights in addition were secured from the city for building a low-level outlet sewer to the East River, 6 ft in diameter, situated in 46th Street, for the drainage of the far-down level of the terminal of which the lowest point was about 80 ft below the crown of the 45th Street viaduct.

Contract and Company Work and Quantities Involved at Terminal.—About seventy-three Grand Central Station and yard contracts, not including those for the electrification, were let from 1903 to 1907, inclusive. The most important contract (for excavation and masonry in the depression of the yard) continued in existence until May, 1907, when, by agreement, the work was taken over by the railroad and with notable success was continued by company forces under the direction of W. F. Jordan, Assoc. M. Am. Soc. C. E., reporting to the terminal engineer, then George A. Harwood,[17] M. Am. Soc. C. E., who had succeeded Mr. Corthell in 1906 and later became chief engineer of Electric Zone Improvements. The yard quantities to be handled from start to finish, according to the plans as they stood in 1907, approximated 3,000,000 cu yd of earth and rock excavation utilized in four-tracking and yard construction on the north, 260,000 cu yd of masonry, and 100,000 tons of steel. Twenty-five miles of pipes and sewers required change of location or replacement, and 27 miles of track called for laying and ballasting, together with their attendant 762-lever signaling and interlocking plant and the placing of passenger platforms along 6 miles of their length.

Capacity and Scope of Terminal.—At this time the area of both levels of the terminal embraced 66½ acres, or nearly three times that of the original depot and yard. Its plans provided for sixteen platforms served by sixteen tracks and a terminating double-track loop, with a possible rapid-transit connection on the suburban level; and fourteen platforms served by twenty-six stub tracks on the express level, terminating with one exception just north of the north line of 43d Street extended, thus totaling forty-two tracks serving thirty platforms on both levels. In addition to this, on the express level, were twenty-six non-platform tracks terminating at the concourse and at 45th Street, making in all fifty-two tracks on the latitude of 45th Street. Added to the sixteen in the suburban station this made a total of sixty-eight terminal tracks abreast on both levels, apart from miscellaneous storage and other tracks still farther north which brought the maximum number abreast to eighty-one. In all, the plans provided for the accommodation of 1,071 cars, or three times the capacity of the depot as it stood in prior years.

The sizes of the spaces intended for the use of travelers on the express level were 80 ft by 204 ft for the general waiting room and 160 ft by 465 ft for the main concourse, with a cab stand on the west having its outlet to Madison Avenue north of 43d Street. The frontage of the terminal on 42d Street measured 300 ft at the surface and 480 ft underground (see Fig. 11). The maximum width of the upper level yard between Madison and Lexington avenues was 940 ft; that of the lower level, 475 ft.

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[17] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 1700.

In the plan the terminal yard on both levels ended on the north at 50th Street, 2,049 ft from the southerly limit of the terminal at 42d Street. The inner six approach tracks continued into the station south of 45th Street on a 0.42% ascending gradient so as to aid in the stopping of trains, and the outer four, two on each side of Park Avenue, descended on a 2.16% gradient to the lower level, the entire ten main tracks fanning out from the four in the Park Avenue tunnel at 57th Street.

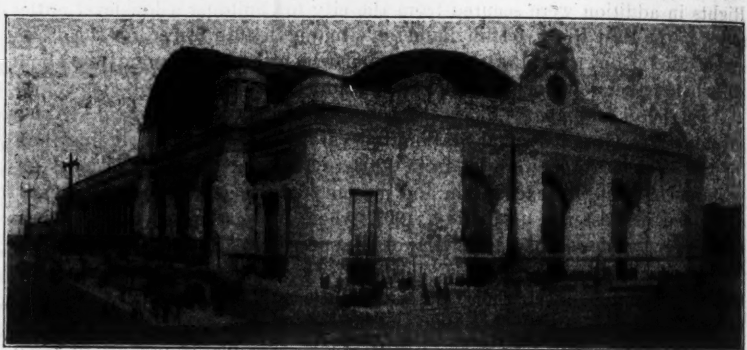

FIG. 11.—PROPOSED TRANSFORMATION, 1907, VIEWED FROM THE SOUTHWEST; ELEVATED DRIVEWAYS ABANDONED TEMPORARILY

Status of Terminal and Associated Work at Close of 1907.—By the end of the formative period in the fall of 1907, the underpinning and side-walls on both sides of Park Avenue south of 57th Street, and the grading between them, had been finished sufficiently to permit the laying of four additional tracks, making eight in all, at the exit of the yard. The major portion of "Bite No. 1" on the easterly side of the terminal area along Lexington Avenue from 50th Street to 43d Street had been completed with its yard, substation, heating plant, express facilities, street viaducts, drainage sewer to the East River, and a temporary passenger station—all in actual use by the New York Central's electrified suburban service. With this had gone the demolition of approximately 200 buildings—a veritable slum clearance—including time-worn and smoke-stained churches, hospitals, and stores. The Annex had been demolished and excavation was in active progress down to the lower suburban level beneath Depew Place and north of it, and the terminal building was nearly completed on Lexington Avenue between 44th and 45th streets. The entire yard, old and new, had been electrified in connection with the use of a third rail of the protected under-control type, devised by the writer in collaboration with Mr. Sprague, and the signals and interlocking were adjusted to the new conditions under which the direction of traffic had been reversed from left-hand to right-hand running. All this work had been done while the daily train service, apart from switching movements, aggregating several hundred in number, had been steadily increasing from about 500 regular trains in 1903 to more than 600 in 1906.

On Madison Avenue, between 43d and 44th streets, a temporary post office and office building had been erected on what was to be the site of the Biltmore Hotel. To facilitate the acquiring of real estate and the handling of rentals, a subsidiary company had been organized, as had been suggested in the parent letter of December 22, 1902. In area available for train operation, the original of a million square feet had been increased to nearly a million and a half, and the car capacity from 374 to 519 cars, or 40%. Moreover, through the substitution of self-moving, multiple-unit electric cars for steam locomotive-hauled trains in the suburban service, after 1906, the total number of train movements in the terminal had been reduced approximately 30%.

In short, all was in order for the continuance of the terminal work as planned, awaiting only a final decision as to the type of station to be built south of 45th Street between Vanderbilt Avenue and Depew Place, and the possible purchase of the easterly portion of the block on the west side of Vanderbilt Avenue between 44th and 45th streets and a full block on 42d Street between Depew Place and Lexington Avenue, which later became the site of the Hotel Commodore, for a further enlargement of the two track levels and the introduction of an additional loop.

Exterior to the terminal the program was well advanced. Four-tracking on the Hudson Division, except at the Spuyten Duyvil cut, was in full use as far as the largely completed new steam and electric terminal at Harmon, N. Y. (South Croton). Two additional main tracks, making six in all, had been laid from Spuyten Duyvil to Ludlow, N. Y., at the south end of Yonkers. Four-tracking on the Harlem Division had been pushed on from Woodlawn Junction to Wakefield, N. Y., just south of Mount Vernon, N. Y., and right of way had been purchased, or nearly so, for its extension to North White Plains where a combined steam and electric terminal had been practically finished, barring the intended loop. With this program, in accordance with the original policy recommended by the writer that the elimination of all grade-street and highway crossings must precede electrification, the raising of tracks for more than 2 miles was in progress in Yonkers, with which was to go the building of three stations and a large freight yard. Similarly, the Marble Hill (N. Y.) Cutoff had been completed with its avoidance of many street crossings and a large saving in distance and curvature. Likewise, there were many other changes of alinement, and the new over and under street crossings with attendant stations on the Hudson Division were already in use or, in a few instances, awaiting an agreement as to plans. On the Harlem Division the plans for two major changes of line were completed, one at Mount Vernon and the other at White Plains, whereby grade-street crossings were to be avoided and sorely needed new passenger and freight facilities provided. At many other places on that division the elimination work with new stations had been completed or was well in hand. All elimination work, and in consequence the electrification on both divisions, north of the limits of New York City, had been seriously delayed by reason of unavoidably long negotiations with the communities affected and, above all, by the inexcusable previous neglect of the state authorities to act promptly on the plans presented to them for approval. Within the City of New York, not only had the main-line work of this nature been completed at

Marble Hill and at the three street crossings immediately south of it, and at Woodlawn on the Harlem Division, but also the work on the Port Morris Branch where a 1,773-ft double-track tunnel had been built under St. Mary's Park in conjunction with many overhead street bridges and the remodeling of its waterfront terminal.

One important feature in the rounding out of the new program was the separation of track grade crossings at the junction of the four-track main line of the Hudson and Harlem divisions at Mott Haven. Plans for this program, with a new station at 149th Street and a loop just south of it, were prepared and the necessary additional right of way purchased, with the idea that the suburban service, turned there as might be found desirable, would be operated at short intervals with varying lengths of trains to promote the growth of Westchester County, New York, and incidentally build up the Bronx as a satellite community to relieve future congestion in Manhattan. It seemed that such a course would accord with sound city planning and add to the safety of operation. Although it had then been authorized by the company, the plan has not been carried out in full, due, it is said, to financial and operating considerations.

Turning now to the subject of electrification, which has been treated in detail by the writer in a previous paper,[18] the two power stations had long since been completed in the fall of 1907, as had also the necessary substations, transmission lines, third-rail working conductors, repair facilities, signaling and interlocking, reversal of traffic to right-hand running, equipment consisting of electric locomotives and cars, standard plans, operating rules and instructions, trains schedules on which to base calculations for the future needs, and temporary electric and steam terminals at High Bridge, N. Y., 7 miles out on the Hudson Division, and at Wakefield 13 miles out on the Harlem Division. Beyond the temporary termini it was not then possible to extend the electric service for which the work was well in hand or completed, because of the delay in the approval of the plans for the grade-crossing eliminations to which reference has been made. Nevertheless, everything was in order to make the final touches to the electrification in the outer reaches of the electric zone as soon as these obstacles to safe operation were removed.

The time to test the new motive power, and the readiness of the Grand Central Terminal to handle it, came on September 30, 1906, when the first electric train was operated into the station with complete success. It was not until December 11, 1906, however, that regular electric service was inaugurated on a small scale, followed by its gradual expansion until, on July 1, 1907, the situation stood as shown in Table 1. In addition, there were the movements of construction trains which went to swell the total, and of switching locomotives still of the steam variety. So far as the New York Central company was concerned, under its contract with the City of New York, the use of steam locomotives in the Park Avenue tunnel was practically ended (96%) a year ahead of the agreed time, July 1, 1908. The delay in the conversion of the New Haven's service was due to its belatedness in its own territory beyond

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[18] "The Electrification of the Suburban Zone of the New York Central and Hudson River Railroad in the Vicinity of New York City," by William J. Wilgus, *Transactions*, Am. Soc. C. E., Vol. LXI, December, 1908, p. 73.

Woodlawn Junction. Not until August 25, 1907, did it become possible to put the reversal of traffic from left-hand to right-hand running into effect.

It is by no means pleasing to record that while the change was in progress an electric train was derailed with fatal results on a curve near Woodlawn on February 16, 1907. Bearing, as he did, the responsibility for having instituted and directed the Grand Central and other improvements in the electric zone, the writer very naturally was called upon by President Newman to defend the electric installation from the charge or suspicion that it was the cause of the wreck. This was successfully done to the satisfaction of Mr. Newman, W. C. Brown, the senior vice-president in charge of operation, the city authorities, the technical press, and the Board of Railroad Commissioners of the state.

TABLE 1.—REGULAR ELECTRIC TRAIN SERVICE AS OF JULY 1, 1907

Description (1)	NUMBER OF TRAINS			Percentages of electric trains (5)
	Steam (2)	Electric (3)	Total (4)	
Scheduled trains, New York Central and Hudson River Railroad	0	240	240	100
Shop trains to and from Mott Haven Yard	15	99	114	87
Subtotal	15	339	354	96
Scheduled trains, New York, New Haven and Hartford Railroad	122	30	152	20
Total	137	369	506	73

In the execution of the work some ninety-seven contracts were let for the electrification, two of unprecedented size for signaling and interlocking, and twelve for electric zone improvements, in addition to the seventy-three at the Grand Central Terminal of which mention has been made. In all, as will be seen, some one hundred and eighty-four contracts were let for the work situated between 42d Street and the northerly termini of the region affected. In addition to this was the vast amount of work planned in detail under the direction of Mr. Harwood and V. Spangberg, designing engineers, and Mr. Katté, electrical engineer, and performed by company forces under the direction of George A. Berry and L. H. Byam reporting successively to the late Henning Fernstrom, M. Am. Soc. C. E., chief engineer, and W. H. Knowlton, principal assistant engineer, outside of the Grand Central zone, and under Mr. Jordan and James C. Irwin,[19] M. Am. Soc. C. E., within it. This included the installation of transmission lines and third rail; grading, laying, ballasting, and bonding tracks; digging and lining tunnels; erecting buildings, retaining walls, bridges, and fences; measures for reversing the direction of traffic and other signal work; snow and wrecking equipment; motive power facilities; and the disposal of excavated material by train from the Grand Central Terminal to outlying points.

The entire work by contract and by company forces had to be done in conjunction with the uninterrupted operation of an extremely heavy express and suburban service centering at the Grand Central Terminal, where the number of train and switching movements, as has been explained, amounted to as much as 1,000 on busy days.

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[19] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 105 (1940), p. 1864.

The dovetailing of the work in general was done through a Construction Committee which held in all 86 meetings between February 20, 1905, and October 31, 1906. On it were the heads of the various activities, including, at different times, Messrs. Fernstrom, Reed, Knowlton, Katté, Corthell, Harwood, Berry, Elliott, Ames, Hoff, Holst, and Spangberg of the engineering department, and A. T. Hardin and Ira A. McCormack of the operating department, the writer acting as chairman.

Estimated Cost of the Enterprise.—By the close of the formative period the estimated cost of the enterprise within the electric zone, terminating at 42d Street, as shown in Table 2, had increased from the original $43,460,000 to $72,000,000, exclusive of additional land purchases at the terminal and at the Marble Hill Cutoff, revenue producing adjuncts other than the projected Biltmore Hotel and space in the terminal building itself, and interest on money during construction.

TABLE 2.—ESTIMATED COSTS EXCLUSIVE OF INTEREST

Description	Original Estimate, May 6, 1903		Modified Estimate, November 3, 1906		Increase	
	$	%	$	%	$	%
Grand Central Station (Railroad).....	8,500,000	20	14,000,000	19	5,500,000	65
Grand Central yard..........................	6,750,000	15	16,500,000	23	9,750,000	144
Electrification..................................	10,400,000	24	14,500,000	20	4,100,000	39
Four-tracking, etc............................	7,810,000	18	17,000,000	24	9,190,000	118
Revenue producers (approximately)...	10,000,000	23	10,000,000	14	0	0
Total............................	$43,460,000	100	$72,000,000	100	$28,540,000	66

It will be seen that the estimated cost of the combined railroad station and yard had increased in its proportion of the total for the entire project from 35% to 42%; that of the electrification had fallen from 24% to 20%; that of the four-tracking, with which went the elimination of grade crossings and rectifications of line and new stations, had increased from 18% to 24%; and that of the revenue producers had decreased from 23% to 14%. The latter did not include an estimated expenditure of many times as much for the tremendous growth of superimposed structures that ultimately was to follow—a growth that in time was to reach $120,000,000 in contrast with the modest beginning of $10,000,000. In all an investment of some $185,000,000 for improvements in and above the terminal and along its approaches, exclusive of interest during construction and land, was to result from the parent thought that by such means vast wealth was to be mined from the air as truly as from "the reefs of the Rand."

GRAND CENTRAL TERMINAL TRANSFORMATION—CONCLUDING PERIOD FROM 1908 TO 1913

Partial Reversion to Elevated Driveway Plan.—It is a pleasure for the writer to record that the circumscribing elevated driveway plan, with its southerly extension over 42d Street, came to life again in 1909, although only the westerly portion was built then, with a short descending grade to reach the level of 45th Street. The latter provision had been made in the original plan of March, 1903, so that the "Court-of-Honor" idea later might be put into effect, should it be concluded that the Park Avenue overhead crossings of 45th Street and the streets beyond were worthy of adoption.

Expansion of Track Layouts, Added Loop, and Station Rearrangements.—The subsequent purchase of additional land on Vanderbilt Avenue and on 42d Street between Depew Place and Lexington Avenue, to which reference has been made, led to the adoption of a plan for an additional loop, two storied, in 1909. At the time, this was particularly desirable for the use of the New York, New Haven and Hartford's locomotive-drawn suburban trains which, unlike the New York Central's multiple-unit trains, could not be properly handled in the lower level with its short-radius loop.

Moreover, this additional purchase, and that of property on the west side of Park Avenue between 48th and 51st streets, made it possible to plan an advancement of the throat of the west yards on both levels two blocks north to 52d Street, and in the station the expansion westerly resulted in an increase in platform space. Accompanying these changes in plan were others for the rearrangement of the approach tracks on the westerly side of Park Avenue and at their entrance at 57th Street, together with the steepening of the gradients to and from the lower level from 2.16% to 3% on the two descending tracks and to 2.7% on those ascending.

The northerly advance of the yard throats made it possible to do likewise with the southern end of the stub tracks on the upper level, thereby facilitating the rearrangement of the plans for the waiting room and concourse, and other changes in the station layout, in accordance with the final understanding reached by the presidents of the three railroads as provided for in the amended Triparte Agreement of which mention has been made. The results of these modifications are shown in Table 3 in which the data obtained in the original Grand Central Station of 1899 are compared with those applicable to the plan as it stood at the end of the formative period in 1907, and with those actually in being in 1925.

Thus, it will be seen that an addition of 19% to the terminal area as it was planned in 1907 made it possible further to increase the track mileage 23% and the platform car capacity 32%, although the total car capacity remained about the same. Compared with the original station of 1899, the capacity of the new terminal, in round terms, thus may be said to have been more than trebled. The area of the station building itself had not been increased greatly, but its facilities, measured by square feet, under both the plans of 1907 and what followed, had been quadrupled. The principal change made in their final proportioning was the decrease in the surface dimensions of the waiting room and its accessories, and of the concourses, as compared with what was proposed during the formative period.

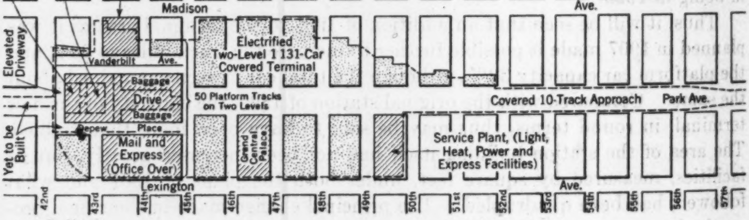
Another reversion to the original recommendations of the writer, in addition to that of the elevated driveways, and an equally happy one, was the abolition of gas for car lighting on express trains in the terminal. The attempt had been made during the formative period to bring this about in favor of electric lighting in all cars, express as well as suburban; but there had been opposition in some directions which long postponed what was clearly in the interest of safety.

TABLE 3.—COMPARISON OF COSTS

Description	Original station, 1899[c]	Grand Central Terminal[d]					
		As planned, January 1, 1908			As built, March 28, 1925		
		S	*E*	*T*	*S*	*E*	*T*
Area of terminal (acres)	23.24	24.22	42.29	66.51	31.21	47.89	79.10
Length of tracks (miles)	10.84	9.81	17.14	26.95	13.79	19.30	33.09
Number of Tracks:							
Abreast (maximum)	....	29	52	81	37	41	78
Against platforms	19	16	26	42	17	33	50
Number of platforms	....	16	14	30	14	17	31
Car Capacity (Number of Cars):							
Platform tracks	214	171	252	423	167	392	559
All tracks	366	389	682	1,071	502	629	1,131
Area of station building (acres)[a]	5.90	....	....	8.10	....	....	6.55
Area of Station Facilities (Sq Ft):							
Waiting room and accessories	19,922	....	....	103,295	....	....	66,411
Concourses	14,814	....	....	157,140	....	....	80,844
Baggage rooms	33,315	....	....	71,400	....	....	57,501
Cab stand and drives	2,952	....	....	39,640	....	....	25,493
Offices[b]	129,500	....	....	475,000	....	....	503,025
Post office and railway mail	33,000	....	....	104,495	....	....	154,642
Total area of station facilities	233,503	....	....	950,970	....	....	887,916

[a] At the street level. [b] Exclusive of corridors and toilets and New York Central railway building of 1925. [c] No suburban level. [d] Column headings: *S* = suburban level; *E* = express level; and *T* = total, both levels.

Opening of the Transformed Terminal.—The new terminal in its final form, apart from developments that were still to come, was opened to the public on February 2, 1913, nearly fourteen years after its birth throes in 1899, and slightly over five years after its formative period had been brought to an end in the latter part of 1907. Referring to Fig. 12, the elevated driveways had been completed on the south and west sides of the station and held in abeyance



Waiting Room
Concourse
Hotel Biltmore (Under Construction)
St.
St.
St.
St.
St.
St.
St.
St.
St.
St.
St.
St.
St.
St.
St.
St.
Madison
Ave.
Elevated Driveway
Vanderbilt
Ave.
Electrified Two-Level 1 131-Car Covered Terminal
Baggage
Drive
Baggage
50 Platform Tracks on Two Levels
Covered 10-Track Approach
Park Ave.
Yet to be Built
Depew
Place
Mail and Express (Office Over)
Grand Central Palace
Service Plant, (Light, Heat, Power and Express Facilities)
Lexington
Ave.
42nd
43rd
44th
45th
46th
47th
48th
49th
50th
51st
52nd
53rd
54th
55th
56th
57th
To 40th St.

FIG. 12.—GRAND CENTRAL TERMINAL, OPENED FEBRUARY 2, 1913, WITH ELEVATED DRIVEWAYS PARTLY RESTORED

on the east side and north of 45th Street. The original idea was finally realized in full in 1927–1928, except that its northerly limit was fixed at 46th Street and its "Court of Honor" narrowed to the width of Park Avenue—140 ft. Provision for a future high office building was made on the line of 43d Street, surrounding the station concourse, instead of a little farther south as originally planned. In this concluding period the work had been directed by George W. Kittredge, Hon. M. Am. Soc. C. E., chief engineer of the New York Central system, assisted by the heads of the engineering and construction organization who, in large part, had been in charge of the work during the formative period. Those in responsible charge were Mr. Harwood, chief engineer of electric zone improvements, assisted by Walter L. Morse, M. Am. Soc. C. E., terminal engineer, W. J. Thornton, designing engineer, E. D. Sabine, resident engineer, and Mr. Jordan, manager of construction; also Mr. Katté, as chief engineer of electric traction, and Azel Ames, W. H. Elliott, and H. S. Balliet, successively signal engineer. In cooperation with them was Mr. Reed, executive of the associated architects until his death in 1911, after which Warren and Wetmore took his place. Mr. Harwood later was to become a vice-president of the company, as was Richard E. Dougherty, M. Am. Soc. C. E., whose early years in his profession were spent on the Grand Central Terminal approaches.

COMPLETED GRAND CENTRAL TERMINAL AND APPROACHES FROM 1913 TO 1939

Full Reversion to Elevated Driveway Plan.—The elevated driveway at the station was, in the first instance, built on its westerly side only, with a grade connection with Park Avenue at 45th Street. Many years after this it was completed as had been planned on the easterly side as well, and both driveways were then extended over 45th Street to a junction with Park Avenue at 46th Street. At least in part the plan of March, 1903, was thus made a reality. Fig. 13 shows an electrified, two-level, covered terminal, with superimposed revenue producers south of 52d Street and 50th Street. Also shown are the restoration of Park Avenue cross streets south of 56th Street as far south as 46th Street, and the elevated driveways over the restored 45th Street and around both sides of the station to a junction with the original Park Avenue at 40th Street. The dream of Mr. Reed in furtherance of the writer's concept of an elevated extension of Park Avenue southerly from 45th Street, ap-

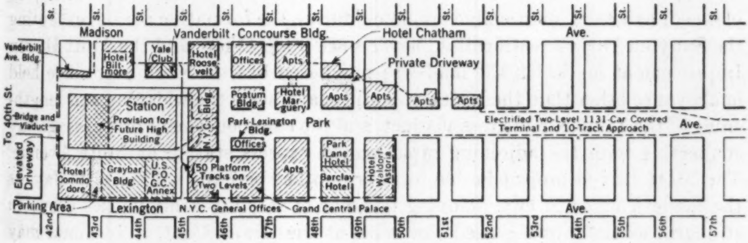

St.
Madison
Vanderbilt · Concourse Bldg.
Hotel Chatham
Ave.
Vanderbilt Ave. Bldg.
Hotel Biltmore
Yale Club
Hotel Roosevelt
Offices
Apts
Private Driveway
To 40th St.
Postum Bldg.
Hotel Marguery
Apts
Apts
Apts
Apts
Bridge and Viaduct
Station
Provision for Future High Building
N.Y.C. Bldg.
Park-Lexington Bldg.
Park
Electrified Two-Level 1131-Car Covered Terminal and 10-Track Approach
Ave.
Offices
Elevated Driveway
Hotel Commodore
Graybar Bldg.
U.S. P.O. G.C. Annex
150 Platform Tracks on Two Levels
Apts
Park Lane Hotel
Barclay Hotel
Hotel Waldorf-Astoria
Parking Area
Lexington
N.Y.C. General Offices
Grand Central Palace
Ave.
42nd
43rd
44th
45th
46th
47th
48th
49th
50th
51st
52nd
53rd
54th
55th
56th
57th

FIG. 13.—GRAND CENTRAL TERMINAL IN 1939

proved at the time by Mayor Seth Low and Nelson P. Lewis, on behalf of the city, and by President Newman, on behalf of the railroad company, thus in principle had come true. It was to the combined genius of Mr. Harwood and Ira A. Place, and of Amos Schaeffer,[30] M. Am. Soc. C. E., on behalf of the city, that fulfillment of the original driveway plan was finally brought about in 1927–1928, a quarter of a century after its inception and 19 years after work on its westerly section had been initiated from 45th Street to 40th Street.

Superimposed Civic Center.—It will be recalled that with the coming of electricity as a motive power, and the opportunity thereby presented for the enjoyment of air rights which until then necessarily had lain fallow, it was proposed that buildings should be erected over the terminal that would produce revenue. In fact, the steel columns beneath had been designed sufficiently strong to serve that purpose. Gradually, in time, primarily under the guidance of Mr. Newman, this had been brought about to a degree that has far exceeded the fond expectations of the writer. Over the tracks have risen such world-known hotels as the Biltmore, the Commodore, the Roosevelt, the Waldorf-Astoria, the Park Lane, the Barclay, and the Chatham; such outstanding office structures as the Graybar and New York Central buildings and the overhead portion of the Grand Central Terminal buildings; such erections intended for special purposes as the Grand Central Palace, the Yale Club, and the U. S. Post Office buildings; and stately rows of apartment buildings of the highest class along Park and Lexington avenues as far north as 50th Street, and along Madison Avenue and Vanderbilt Avenue as far north as 48th Street and 49th Street, respectively (see Fig. 14). The Grand Central Zone has become a self-contained city clearly evident to the casual onlooker who little knows that beneath it are the terminal yards of two great railroad systems.

Approaches Within New York City.—The work on the approaches, as planned, had been practically completed during the formative period, excepting the Spuyten Duyvil rectification, never since undertaken, and the Mott Haven Improvement for which the land was purchased but the work ever since held in abeyance other than the laying of additional tracks, the widening and lengthening of the 149th Street cross viaduct, and the commencement of a passageway connecting with the adjoining rapid-transit express station at Mott Avenue. The Mott Haven project in one form or another perhaps will have its day, as the modern move in city planning in favor of satellite communities grows in strength, as the growing needs for relief at the Grand Central Terminal may prompt the stopping of a portion of the suburban service at an outer point such as this, and as the demands of the public for the stoppage of express trains in a borough of a million and a half population become more insistent.

Approaches in Westchester County.—On the Hudson Division the improvements as planned were completed in the instances where that had not already been done during the formative period; but on the Harlem Division the four-tracking was completed to Mount Vernon only. Beyond that the laying of the intended additional two tracks and the construction of the wye at North White Plains have not been completed; but the new stations and grade crossing eliminations in that territory have been finished with due regard for the future four-tracking as planned. It is to be expected that the continued remarkable growth of Westchester County, particularly by reason of its character, will in time call for the rounding out of the improvements as originally designed.

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[30] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1689.

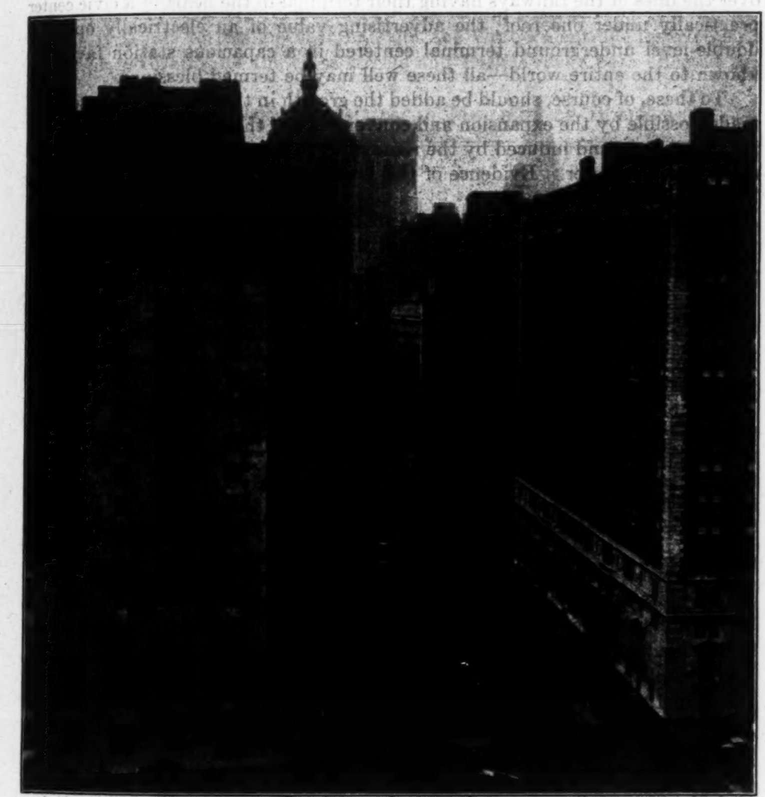

Fig. 14.—View on Park Avenue, Looking South from 50th Street in 1939, Showing Revenue Producers Over the Terminal and the New York Central Building in the Background, Through Which Connection Is Made with Elevated Driveways Surrounding the Station Beyond (Compare with Figs. 3, 7, and 9)

Beneficial Results

Railroad Blessings.—To the railroads using the transformed terminal and its approaches have come numerous blessings. Safety of train operation through the abolition of smoke and gas in the Park Avenue tunnel; adequacy of facilities for the efficient handling of a constantly increasing passenger service; the minimizing of "dead" train haulage to and from Mott Haven and of switching in the terminal yard; economy of operation as demonstrated in the writer's paper[18] of which mention has been made and set forth in reports made to Mr. Newman in August and September, 1907; revenue gained on a vast scale from superimposed buildings in the Grand Central zone; the inducement of travel over the lines of the railways having their terminal in the heart of a civic center practically under one roof; the advertising value of an electrically operated double-level underground terminal centered in a capacious station favorably known to the entire world—all these well may be termed blessings.

To these, of course, should be added the growth in the number of passengers, made possible by the expansion and convenience of the improved terminal and its approaches, and induced by the removal of deterrents incident to the use of steam motive power. Evidence of this is shown in Table 4.

TABLE 4.—PASSENGERS IN AND OUT OF THE GRAND CENTRAL TERMINAL (IN THOUSANDS)

Description	1906		1930		Percentage of Increase	
	Commuters	All	Commuters	All	Commuters	All
Hudson Division	3,065	5,829	9,913	14,605	223	151
Harlem Division	3,570	4,905	16,351	19,160	358	201
Total New York Central and Hudson River Railroad*	6,635	10,734	26,264	33,765	296	215
New York, New Haven and Hartford Railroad	3,177	8,305	9,444	16,878	197	103
Grand total	9,812	19,039	35,708	50,643	264	166

* Including New York and Putnam and 125th Street stations.

In a word, the traffic of the New York Central road in 24 years had quadrupled in its commuter service and more than trebled in its combined commuter and express service, whereas that of the New York, New Haven and Hartford had trebled and doubled, respectively. For both roads in 1930 there was almost a quadrupling of the number of passengers that had been moved in commuter trains in 1906 and something less than three times the number handled in the trains of both services. The new electrified terminal with treble the car capacity of its steam predecessor, or quadruple the capacity if weight is given to the diminished need for switching through the use of electric motive power, had, therefore, fully measured up to the needs of the occasion in 1930. Taken all in all, so far as the writer is aware, the enormous expenditures made on the terminal and its approaches have been fully justified by results, judged from the viewpoint of the railroads concerned.

Public Blessings.—Steam railroads almost invariably have a depreciating effect on neighboring property, especially in urban and suburban communities where their smoke, gas, cinders, noise, and unsightliness in the open discourage civic development and are productive of shabby surroundings or even slums. There was no more striking evidence of this than in the vicinity of the original Grand Central Depot yard and therefrom northerly along Park Avenue as far as 96th Street. If proof of this were needed, resort may be had to a comparison of the assessed values of real estate in 1904 before the intent to transform the terminal had become widely known and in 1930 when completed (see Table 5).

It will be seen that the increase in Manhattan as a whole was 175% (174.7% to be exact) and in Section 5, exclusive of the Grand Central zone and beyond, 168%. It is fair to assume that had not the Grand Central improvements with their restoration of the cross streets and Park Avenue been effected the increase in assessed values in the section of which they are a part would have been no greater than its remainder, namely 168%. The chances are that it would have been less because of the continued depressive effect on property facing a smoky, noisy, unsightly terminal and the open craters along Park Avenue from whence smoke, gas, and cinders were belched. However, on the more conservative assumption that, were it not for the terminal improvements, the situation would have corresponded to what took place in the borough as a whole, the increase in assessed values attributable to the transformation may be stated as shown by items 5, 6, and 7, Table 5.

TABLE 5.—ASSESSED VALUES, IN MILLIONS OF DOLLARS

Item	Regions	1904	1930	INCREASE	
				$	%
1	New York City	5,015	19,717	14,702	293
2	Borough of Manhattan	3,677	10,102	6,425	175
	Section 5 (Block 1,280):				
3	Within the Grand Central zone[a]	700	2,427	1,727	247
4	Beyond the Grand Central zone[b]	432	1,159	727	168
	The Grand Central Zone and Beyond:[c]				
5	Actual	268	1,268	1,000	374
6	Computed at the same rate of increase as Manhattan (174.7%)	268	735	467	175
7	Increase attributable to the transformation	0	533	533	...
	Westchester County:				
8	Actual	193	1,758	1,565	813
9	Computed at the same rate of increase as New York State as a whole	193	796	603	313
10	Increase attributable to causes arising from the transformation	0	962	962	...

[a] Between 40th and 96th streets from the East River to Sixth Avenue as far north as 59th Street; then to Fifth Avenue as far north as 96th Street. [b] Exterior to the Grand Central zone and beyond, to 96th Street. [c] Between 42d Street and 96th Street and between Madison Avenue and Lexington Avenue (assembled through the kindness of Harry P. Moran, Stanley Skarvan, and Frank Klopf, of the U. S. Works Progress Administration).

The result to the people of New York in enhanced assessable values because of the Grand Central improvements, therefore, may be taken at something in excess of $500,000,000 and yielding in taxes at the current rate of 2.7% more than $14,000,000 per annum. The incalculable indirect effect of the change on more distant property has also been very great.

In a similar manner it is possible to view the situation in Westchester County as shown by items 8, 9, and 10, Table 5. It would seem fair to demonstrate in this way that in Westchester County there has been an increase in assessed values of nearly $1,000,000,000 in excess of what would have occurred if the change of motive power and other railroad improvements had not been made. The elimination of steam locomotive nuisances, and of grade-street crossings with their accompaniment of danger and delay, certainly had a pronounced effect on the attractiveness of the region as an abode for city workers.

With this went a more frequent train service, the freeing of passengers from terror in the passage of the Park Avenue tunnel, and the convenience and protection from the weather that has come from affording a means for circulation by the travelers under cover in the midst of a new civic center supplied with adjacent express stations of the city's rapid-transit systems.

If further proof of the beneficial effect of the change of motive power is required, it may be found in the marked difference between growths of population, travel, and assessed values of real estate in the commuter regions in which that change has come about north and east of New York City, and in the commuter region west of the Hudson River where such a change had not been effected in 1930.

TABLE 6.—COMPARISON OF PERCENTAGE INCREASES—COMMUNITIES SERVED BY STEAM TRAINS AND BY ELECTRIC TRAINS

Item	Description	Steam[a]	Electricity[b]	
			Westchester County	Long Island
1	Population: 1930 over 1900	142	183	249
	Passengers Handled:			
2	1930 over 1911	69	....	292
3	1930 over 1906	....	166	....
	Assessed Value of Real Estate:			
4	1930 over 1911	....	440	597
5	1930 over 1914	161	....	....

[a] Percentage increases in the steam-operated region west of the Hudson River. [b] Percentage increases in the electrically operated regions north and east of New York City.

From Table 6 it will be seen that in New Jersey and New York west of the Hudson River the increase in assessed valuations was only a fraction of that in Westchester County (161% against 440%) and the disparity is still greater when the comparison is made with Long Island—161% against 597%. The comparisons, of course, are not exact, as the earlier dates differ due to an absence of obtainable data for corresponding years; but they are believed to be sufficiently accurate to warrant the drawing of general conclusions for the purpose of illustration. Unquestionably, the pronounced increase in property values in Westchester County is largely, if not entirely, to be ascribed to the impulse that had its start in the change of motive power and associated improvements. Had there been no change, it is fair to assume that the outcome would have been no more happy than it was on the westerly side of the Hudson River.

Then, too, it is believed that the change of motive power and other improvements in the railroad service in Westchester County was the spark that led to the initiation of parkways and parks in that region from which gradually came forth a system of pleasure thoroughfares and recreational facilities of which there is none more beautiful in the United States.

Assuming that the new Grand Central Terminal and its associated changes for the better were the primary cause of the phenomenal increase in assessed real-estate values in Westchester County, the resulting indirect benefit to the public at the current tax rate of 2.706% is in excess of $26,000,000 per annum.

RÉSUMÉ AND CONCLUSION

The Project in Perspective.—In this paper the attempt has been made to portray a scene in which the parent reasons for doing things, woven on a background of early history, would invest the rounded image with proportions which, seen from a distance, may not be distorted.

In the mind of Commodore Vanderbilt was born the idea of a new terminal on a 40-yr-old ramshackle entrance line in New York City, to which was to be brought the traffic of a great system of railroads under his control. Given life by him, the resulting Grand Central Depot, with its approaches, continued to function through three succeeding epochs of reconstruction and enlargement, until after twenty-eight years, in 1899, the use of steam locomotives in and approaching it had become insufferable and its design and capacity had become unsuited for the demands that had come upon it; sixty-eight years had now elapsed since the owning company had made its first bow as a horse railroad in 1831.

The inescapable substitution of electricity for steam as a motive power (experimental though it was for trunk-line service), combined with the idea, born in 1902, that revenue plucked from the air might be used to finance its tremendous cost, had its fruit in the inception of a new Grand Central Terminal with the electric service extended to the limits of the suburban zone. Following this step came ten years spent in development of plans, the construction of the project, and the opening of the transformed terminal to public use on February 2, 1913.

Thenceforth the project was rounded out and blessings reaped which have their evidence in the multiplied travel and revenues enjoyed by the New York Central and New York, New Haven and Hartford systems; in the replacement of slums in the heart of the City of New York by a remarkable civic center; and the giving of an impulse to the development of Westchester County, which, in the form of taxes, is bringing some $40,000,000 per annum to the public purse.

The Future.—Attention has been directed to the trebling of the number of passengers handled in the terminal since 1906, and to its capacity which likewise has been more than trebled. It would seem, therefore, that the time is not far distant when, as was done forty years ago (1899), thought must be given to the taking of measures for handling a further increase of traffic and for satisfying public demands. What may have been done in this direction the writer is unaware. The installation of a satellite terminal at or near the Mott Haven yard in the Bronx would seem to be worthy of consideration as one way out; other suggestions are the provision of enlarged facilities in the Grand Central zone in connection with a new crosstown link to the Hudson Division main line on the West Side, as was proposed by the writer during the formative period, or a new tunnel beneath the present viaduct and tunnel in Park Avenue, or in part beneath adjoining thoroughfares as has been suggested by the Regional Plan Association of which H. M. Lewis, M. Am. Soc. C. E., is chief engineer and secretary. It is to be hoped that a possible solution for this problem may have public discussion without waiting for the pressure of events such as made the railroad companies' lot so painful in the closing years of the nineteenth century.

ACKNOWLEDGMENTS

To the New York Central System the writer owes his thanks for permission to deposit his voluminous duplicate company records in the New York Public Library, for certain of the photographs herein used, and for up-to-date plans of the terminal. It is perhaps needless to add a further expression of the writer's gratitude to those who were associated with him in the remodeling of the old steam-operated Grand Central Station and in the formative years of its transformation into its electrically operated successor which became known as the Grand Central Terminal; also to those of the Tax Department of New York City and of the U. S. Works Progress Administration who gave of their time in the assemblage of assessed values. The names of those who were more prominent in the carrying of responsibility in these respects are given in the body of the text; the names of the others as a rule are given in the technical press of the day.

The very nature of this paper has required a rather complete statement of the persons involved. Such mention, however, has been conservative and in full recognition of the Society's rule against discussing personalities in its technical papers. It is expected that discussers, likewise, will confine themselves to the technical problems, and not the persons, as the subject of their comments.

Copies of the manuscript in full, from which this paper has been prepared, including illustrations, bibliography, and laymen's drawings, may be consulted at the Engineering Societies Library[21] in New York, the New York Public Library, and the Congressional Library in Washington, D. C.

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[21] 29 West 39th Street, New York, N. Y.

DISCUSSION

F. Lavis,[22] M. Am. Soc. C. E. (by letter).—This all too brief review of the events and processes leading up to the final development of the present Grand Central Terminal in New York is a paper of the kind that engineers need so much and so seldom see. It is an account of particular interest to all who look beyond surveys and the calculation of the size of an I-beam, or even of an unprecedented bridge or other structure, to the fundamental effect of engineering structures and engineering design on economics and on public and civic affairs. The author is to be congratulated on having written and on being able to write such a paper.

One hesitates to criticize it, therefore, but knowing the author for what he is, a passionate servant of the truth, the writer believes that he will accept a very slight amendment to his statement about the effect of this improvement on assessed valuations in Westchester County.

That the terminal improvement and the electrification of the railways within the northern suburban area of New York City have contributed to the building up of Westchester County no one will deny; but it is to be questioned if this expansion can be attributed to this improvement to the extent indicated in the paper. Assessed valuations in Westchester were increased, materially and arbitrarily, in 1920 and 1921. In Scarsdale, for instance, assessed valuations were doubled at that time, from about $10,000,000 to $20,000,000, with only a very small increase in actual values. Similar increases occurred throughout the county at about the same time in an attempt to comply with the law which requires assessments to be equal, as nearly as possible, to the full value of the property.

In attempting to show that the rate of increase in Westchester County is very much greater than the rate of increase in the state as a whole, it must be borne in mind that Westchester is the nearest available suburban area to New York, the largest and richest city in the world. Long Island and New Jersey are separated from Manhattan by river barriers, whereas Westchester has easily available land connections and the railways serving it have a terminal in the heart of New York City which, during the period considered by the author, was additionally connected to downtown New York by the first subway line. The growth of Westchester has been coincident with the growth of New York City, both in wealth and the number of inhabitants, and must be attributed in part to this growth of the city.

During the period under consideration, Westchester was also building the Bronx River Parkway, the first of that series of great highway arteries which have made the county so easily accessible; and as a matter of fact the sponsors of this parkway have claimed that it was the large expenditure of county money for this purpose that has been responsible for the great rise in land values and assessments in the county. Certainly these parkways are responsible for helping to make Westchester a desirable place in which to live.

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[22] Cons. Engr., New York, N. Y.

In spite of these facts, however, the author has ample claims on the gratitude of the profession in describing a remarkable piece of work very well done. From his own experience, the writer can testify to the competence of the organization which did this, handling millions of passengers while the work was being completed with scarcely any delay or inconvenience.

So far as Westchester is concerned, even though it may not be conceded that its growth was wholly due to the terminal improvement and the electrification, it must be conceded that the two went hand in hand—that a great public utility had vision and measured up to its duties and responsibilities.

E. R. Hill,[23] **M. Am. Soc. C. E.** (by letter).—The traveling public takes for granted today the two major passenger stations of the railroad on Manhattan Island. The generation is passing that saw the building of the Pennsylvania Station and of the present Grand Central Terminal, and it is therefore timely and fitting that the full story of the latter should be spread before the members of the Society.

This station presented many problems differing from those connected with the Pennsylvania Station. This latter was an entirely new project involving tunnels under the Hudson River from New Jersey and a connection by tunnels under the East River with a large new train yard on Long Island. The station, therefore, partook more of the nature of a major stop en route than of a stub-end terminal.

The problem before the New York Central Railroad Company, however, was the rebuilding and expansion of a busy operating terminal station, together with the reclaiming of valuable building sites by the placing of the tracks underground. This involved the adoption of electricity as the means of motive power at a time when electric traction was still in its formative stage.

The difficulties of such an undertaking were enormous, of course, for, besides the physical problems encountered, it must be remembered that all of this work had to be done without interference with the operation of the railroad with its complex combination of through passenger service from the north and west and, for that day, a heavy commuter service. Some of the older commuters will remember the wooden ramps which were built at various degrees of slope to test out just how steep they could be made without slowing up the march of the crowds unloaded from the trains.

The transformation of Park Avenue, then ill-named, from its smoky and generally undesirable character to its present state, also is one of the outstanding features of the project, taken very lightly and casually by the average traveler of today.

That the creators of the terminal built well is indorsed by the fact that, notwithstanding the enormous growth of the traffic handled, the station has continued to do its work satisfactorily with virtually no major changes. The well-planned separation of the through and commuter traffic remains as it was at the outset and handles the tremendous crowds of rush hours and holidays with an efficiency that is the admiration of all visiting railroad operators. Mr. Wilgus' data, given in Table 4 and supporting text, are impressive, the percentage of increase in traffic being 264 for commuters and 166 for all kinds of traffic. These are yearly totals; it is probable that the daily peaks would show even greater increases.

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[23] Cons. Engr. (Gibbs & Hill, Inc.), New York, N. Y.

Mr. Wilgus points out that the time approaches when thought must be given to handling a further increase of traffic. Who shall say what the future holds? Are cities in the United States to grow indefinitely or are they to feel increasingly the effect of decentralization of which there are signs? Whatever is ahead, one can but hope that men will be found of the caliber and character of their predecessors to solve the new problems as they arise.

Alonzo J. Hammond,[24] Past-President, Am. Soc. C. E. (by letter).—In the "Synopsis" of this paper, the author rightly refers to the exceptional advantages achieved in the interest of the owner, and of the public as well, by the utilization of overhead air rights due to the change of motive power from steam to electricity.

The fact that the approach tracks ran through a public street where the steam and smoke were objectionable, as well as in the zone of the terminal proper, made it all the more important to change the motive power—the result of which, of course, was "a clean bill" for the very remarkable development of air rights. If the terminal alone had been considered, the result could have been obtained, as it has been in the case of the Chicago Union Station, Chicago, Ill., where steam is the motive power.

During the earlier stages of construction of the Chicago Union Station, full-scale tests were made over quite a period of time to determine the best type of inlet areas, and the type of fan and power required for removing promptly steam and smoke from a locomotive through ducts to a satisfactory outlet. The results obtained made possible the use of air rights for the Daily News Building and the Post Office, the latter extending for a block about 800 ft long and the full width of the station tracks. It will be admitted that the results do not measure up to those obtained with electric traction, but are fairly satisfactory, with one important exception—the corroding effect of smoke.

The author has been able to add a new term to the classic definition of engineering, and he is to be congratulated not only in having the vision but the opportunity to "evoke" such a magnificent station development. In consequence, he has well earned the retrospect of a job well done.

In connection with his studies for the Chicago Union Station, the writer had occasion to spend quite some time making an intensive study of the Grand Central Terminal, the engineering design, operation, income from facilities, economics of investment, as well as architectural treatment for combined station use and office facilities. The writer was impressed most by the intensive use of vertical space in a two-stage development and the minimum effort required of the passenger to reach the various places he had to go to supply his wants for service—the ticket offices, parcel room, information booth, entrances to the train concourse—all in plain sight and easy of access when he reaches the foot of an easy ramp from the street, or a few short steps after leaving his taxicab. Such a well-planned sequence of movement for the passenger exists in no other station, to the writer's knowledge. Opportunity was there and was taken advantage of to a superlative degree.

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[24] Cons. Engr., Chicago, Ill.

The author has given a very enlightening story of the processes of the Electric Traction Commission in reaching conclusions, not only as to type, but extent of electrification, as, for example, to stop at Mott Haven, five miles out, or go to Harmon, about thirty miles farther. Table 4 shows the increase in commuters from 1906 to 1930, but it would have been of interest if the immediate effect of electrification could have been noted resulting from greater speed and comfort of the suburban trains. This is mentioned for the especial reason that beyond a certain zone of the Illinois Central suburban electrification in Chicago, both on the South Chicago branch and the main line, there was a very prompt reaction in building, and new neighborhoods developed, increasing the local traffic quite promptly.

The architectural compromises referred to and the statement that the intended high office building over the south end of the station on 42d Street "fell by the wayside," lead one to the conclusion that the ultimate result was most fortunate in providing an elevation on 42d Street, facing south into Park Avenue, of matchless beauty and, in addition, the glassed arched gables permit the sun's rays to pour down into the passenger concourse with the result that the people are kept in a cheerful mood when waiting for a train.

Such a development requires many studies to be made and many discarded. In the case of the head house of the Chicago Union Station, there were some thirty studies made. The caissons were put in for the selected type before the United States entered the World War, and afterward a different design was adopted, necessitating a revision of nearly all the caissons. In the case of the Grand Central Depot, it took courage and vision to tear down the old structures and start "from scratch."

Where there is so much to commend, there is left very little to criticize in such a terminal. The writer has found one element, however, which is inherent with a stub-end terminal with passenger platform used for handling baggage and mail. The writer has timed the unloading of baggage and mail onto trucks from an inbound train and has found it to be from 20 to 22 min. In the meantime, the passengers have all gone by on the narrow lane left by the trucks; so, without invidious comparisons, one might refer to the Chicago Union Station's separate platforms for such traffic, a detail believed to be peculiar to that station.

At times there have been criticisms from some sources of the money which railroads have spent for what were termed "monumental" stations. In 1916, when Robert Trimble was preparing his presidential address for the American Railway Engineering Association, the writer gave him an editorial comment by John Henry Zuver in a South Bend (Ind.) newspaper, which ran as follows:

> "In the middle ages, communities built magnificent cathedrals—and paid for them—and who questions the ennobling effects of these rich and spacious structures on all beholders? They satisfied and glorified the lives not only of those who built them, but also of their descendants. The genius of the American people runs to transportation and to business. Why not express through those mediums the artistic soul of the nation? Who that ever stood before the massive pillars of the Northwestern in

Chicago, who that ever walked the floor of the wasteful concourse in the Pennsylvania Station in New York, with its noble space and simplicity, its astonishing and grateful silence, can ever think of them without a thrill?"

H. L. RIPLEY,[25] M. AM. SOC. C. E. (by letter).—In writing the history of the development of railroad service for passenger transportation northerly from New York City, along the Hudson River and into Westchester County and New England, Colonel Wilgus has again done a notable service to the engineering profession and to the public at large.

His description of the peregrinations of the terminal from City Hall Square to the present location of the Grand Central Terminal at 42d Street and Park Avenue would be a worth-while endeavor of itself. His description of the terminal project, however, makes it of vastly more interest and importance to the engineering profession. His description of the terminal and its northerly approach, with maps, tabulations, and photographs, presents the problem in such clear and complete detail that one who has never seen it can form a clear picture of the magnitude of the enterprise as a whole.

He has touched only lightly upon a phase of the matter which, to the writer at least, seems to be the most remarkable part of the undertaking (excepting its original concept), namely, that the entire job was completed under a traffic maintained on the same location, reaching more than 500 regular scheduled trains, and which involved as many as 1,000 separate movements per day.

All engineers know that it is one thing to construct a project "in the open." It is quite another to do the same job in the heart of a great city, under intensive traffic, and to keep that traffic moving without delay or serious inconveniences; and those who had occasion to use the terminal during the construction period know that this was accomplished remarkably well. The excavation had to be made over acres of area down to a depth of approximately 45 ft below 42d Street, still keeping the trains moving "on time."

As the author states, the station and its environment can now be seen "in perspective." The traveler is likely to overlook the influence it has had in adding to the "more abundant life" of the community and to forget the troubles and tribulations involved in its accomplishment unless he remembers the conditions that existed thirty or more years ago, when the two sides of the city were cut apart from 42d Street to 56th Street (see Fig. 5) and the area between, now one of the finest parts of the city, was a smoky waste of tracks and service buildings; or, unless he compares a photograph of that area as of today with one, say, as of 1906 (see Figs. 6, 7, and 14).

The writer wishes to add a word of appreciation and indebtedness to the author for his vision in planning the enterprise, and for his record of the whys and wherefores as an inspiration to others.

A. J. MEEHAN,[26] M. AM. SOC. C. E. (by letter).—As one who, since boyhood, has revered the Grand Central Terminal because of its inherent majesty, the writer is particularly grateful to Colonel Wilgus for presenting a comprehensive story of the inception and development of this daring enterprise.

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[25] Contract Agt., N. Y. N. H. & H. R. R., New Haven, Conn.

[26] Senior Bridge Engr., State Dept. of Public Works, Div. of Highways, Bridge Dept., Sacramento, Calif.

Numerous instances in the history of this terminal have demonstrated how readily alterable it has been to meet new demands. Such adaptability has been recognized as a distinct tribute to the farsightedness of its engineering planning. The success of this huge undertaking continues to serve as an outstanding source of inspiration to other engineers who find themselves confronted with complex traffic and service problems.

The "Synopsis" and "Résumé and Conclusion" of the paper intimate that as yet there are some unsolved future problems in connection with this project. One of the remaining problems suggested itself to the writer the summer of 1940 as he gazed down upon the structure from a building on Park Avenue just south of 42d Street. In particular, it is the traffic situation on the elevated viaduct at Park Avenue and 42d Street, a block plan of which is shown in Fig. 13. This tee-shaped entrance to the viaduct surrounding the building is hazardous because of the right-angle turns and narrow roadways. At this point, the Park Avenue section of the viaduct has one lane of traffic in each direction. As one observes the traffic entering or leaving Park Avenue as it weaves into the opposing lanes in order to negotiate this turn, and likewise notes the effect upon the traffic making the continuous circuit of the terminal on the elevated driveway, he shudders to think of the accident menace and how much greater it must be when the surface is slippery.

This condition could be remedied by replacing this right-angle corner by a flare or curve which would provide a greater turning radius for in-bound or out-bound Park Avenue vehicles. Such a solution would require care to preserve architectural appearances and to maintain the existing under-clearance on 42d Street. However, with the great range of construction materials now available, it would be a comparatively simple matter to effect this improvement in a manner that would satisfy structural and architectural requirements.

J. P. HALLIHAN,[27] M. AM. SOC. C. E. (by letter).—It is not often that the responsible designer of a facility of the importance and complexity of the Grand Central Terminal is afforded an opportunity to compare the performance with the original objectives. Colonel Wilgus has gone further in revealing how the original idea was molded into the finally accepted plan in a period when execution of the principal features involved entrance into almost uncharted fields of engineering and economics.

It is fortunate that the men of finance, of the law, of engineering, architecture, and city planning, assembled on the Commission, were also men of courage and vision, able to see that the influence of a great terminal in a great city extended far beyond its functions as a transport facility.

One of the most impressive features of the plan was the provision for taking care of the surface transportation of the future which, it is not surprising to learn, was regarded as an excessive provision by some of the members of the Commission, and was not fully carried out until some time after the completion of the terminal structure. Had this feature of the plan been surrendered, it can well be imagined what an impossible traffic situation would have been created.

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[27] Director, Municipal Eng. Section, WPA, Washington, D. C.

In the light of present knowledge it is regrettable that the Court of Honor could not have been retained, but in the thought of that day the consequent sacrifice of building space must have made it appear as an inadmissible extravagance.

Colonel Wilgus' review of the deliberations of a commission made up of experts in their several lines, conscious of the necessity of bold pioneering in order to achieve the high purpose of getting the greatest public service within the limitations of an enterprise that must be permanently self-supporting, is a distinct service to the engineering profession.

It is safe to say that there will be no terminal development of the future that will fail to draw inspiration from the story of the Grand Central Terminal, which so effectively serves its purpose after more than a quarter century of use.

ARTHUR V. SHERIDAN,[28] ASSOC. M. AM. SOC. C. E. (by letter).—The evolution of the Grand Central Terminal, for a generation the world's most famous and interesting rail terminus, is a fascinating chapter in the career of some notable men. The talents, efforts, disappointments, perseverances, and travail of many forceful and imaginative minds are as truly a part of the great terminus and its associated projects as are the materials embraced in the monumental center which today invokes the admiration of those who view and the commendation of those who analyze it.

The brief yet adequate manner in which the story of the railroads, whose expansion lies behind the development of the Grand Central Terminal, has been told leaves the commentator little opportunity for any remarks other than an appraisal of a scholarly recording. By his concise presentation of salient facts, his recital of leading personages, their dreams and their rôles, and the exposition of the problems, trials, criticisms, and difficulties encountered en route, the author has produced a treatise interesting and instructive to any one who reads it. Although it is probable that most of the material contained in this paper is available in record form, the task of gathering and compiling it is undoubtedly one of considerable proportions. The manner in which this has been accomplished, including as it does personal records and recollections, gives documentary value to the paper.

It is particularly as a planning achievement that the Grand Central Terminal and its related features merit review. Individual parts of this project have their counterparts in many locations, in some instances even in more advanced forms; but it is doubtful if there presently exists anywhere a composite project more completely and adequately conceived and executed. No plan can be evaluated until it has been consummated. No person deserves the appellation of planner until his plans have been put to the test. In the planning of the Grand Central Terminal, almost every social, economic, and physical factor entering into daily urban life had to be weighed, located and judiciously incorporated in a form intelligible to those entrusted with the engineering, architectural, financial, legal, and administrative features of a travel, residential and business community, all located within one great indoors.

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[28] Planning Commr., City of New York, New York, N. Y.

The writer recalls much of what has taken place during the past 45 years. As a boy in old Kings Bridge, N. Y., he daily watched the trains that sped over the rails passing between Spuyten Duyvil and the Grand Central Station and as a youth he used them en route to seek an education. He still can visualize the cut through solid rock that served as a railroad path for what has since been incorporated into the present pattern of Kingsbridge Avenue, Broadway, and Putnam Avenue, adjacent to West 230th Street and West 231st Street in the Bronx. The rerouting of the railroad, a well remembered undertaking, along the north bank of the Harlem Ship Canal, eliminated dangerous crossings and reduced to a mere memory a once active suburban railroad center jointly served by the New York Central and the New York and Putnam railroads and long since replaced by nearby intensive developments.

It is interesting to note how expansion of the City of New York has paralleled that of the railroad. The early movement northward of the terminus of New York's first railroad followed the trend of population on Manhattan Island. By the time the present Grand Central was in the process of final planning, New York had been greatly augmented by the formation of the Greater City, which brought Kings, Queens, and Richmond counties into the perspective of future plans. The center of population was no longer to move north but rather to the east. In view of its geographical position, 42d Street was well envisioned as a permanent terminus for a railroad connecting New York and the great centers of the east, north, and west. In this, as in other respects of its planning, the project has demonstrated the foresight of its authors.

There can be little question that the local electrification of the railroad was one of the great milestones of its existence. To this accomplishment, both public and corporation can attribute a relationship that has permitted mutual expansion. It is inconceivable that a Grand Central Terminus could have remained where it is, or in fact within the city at all, had former methods of propelling trains been retained. That science and engineering have made possible a speed, cleanliness, smoothness, and safety of travel undreamed of, even a half century ago, is obvious to all who have witnessed the progress of intervening change; nor is the end in sight.

The effect of time, from the viewpoint of finances, upon the execution of a project, is vividly portrayed in the accomplishment presently being reviewed. Great as was the cost of the project planned about 1900 and substantially completed within the decade following, the sum was only a fraction of what would be required to execute it at this writing.

In comparing the original plans with present conditions, one is constrained to the opinion that it is somewhat regrettable that the proposed Court of Honor at the north end of the main station building was eliminated from the project. The esthetic value of the great structure facing Park Avenue would have been considerably enhanced and the approach to the elevated roadways encircling the building would have been less suggestive of a tunnel hazard to an automobile en route.

Of particular interest to the writer and to a community of 1,500,000, known as the Bronx, is the suggestion for a great civic and business center in the vicinity of 149th Street and the New York Central right of way. From the standpoint of public service, it is indefensible that the Bronx should be without facilities for receiving and discharging passengers from cities served by the two railroads, none of which centers, except Chicago, Ill., has a population equal to that of the Bronx and its neighbor Westchester. Included in the original plan of Colonel Wilgus, this project was advanced to a stage which convinced both public and elected officials that certain associated city planning was necessary. Two subway routes now reach the location, a central post office has been erected, Mott Avenue has been widened and renamed the Grand Concourse, and a magnificent civic and county center, with a modern hotel, located within a half mile has been built.

Several years ago, James J. Lyons, President of the Borough of the Bronx, made overtures to the railroad in the hope of reviving the project. When these failed, the City of New York instituted legal proceedings to compel action. That the railroad should be reluctant to proceed with its plan of nearly forty years standing will probably be attributed to financial difficulties. However, when one reflects that the population, which it would serve, has increased from less than half a million to more than two millions, since it was first deemed desirable, it is difficult, even from an economic standpoint, to justify a postponement of so many years. The inconsistency of the railroad policy is particularly confusing to those who, like the writer, recall the directional sign and passageway installed at the instigation of the railroad for the purpose of leading passengers to and from the subway at the Mott Haven terminal station. Barring perhaps the intense concentration of people and business, practically everything that has been possible at Grand Central could be duplicated at Mott Haven.

A proposal for covering the Harlem Branch of the railroad north to Fordham, including the reclaiming of adjacent blocks, has been developed, and a Park Avenue comparable in distance and superior in width and treatment to that north of Grand Central awaits only financial and legal impetus. Preliminary plans and estimates for connecting 96th Street and Park Avenue in Manhattan with the Grand Concourse and with Park Avenue, by means of an express highway, have also been made with the approval of the Board of Estimate. Both of these projects were developed under the direction of the writer as chief engineer of the Bronx and for five years have been part of a borough arterial highway program. Although it is generally conceded that the most desirable treatment for Park Avenue south of the Bronx would be to depress the railroad, it is also recognized that placing the New York Central tracks north of 96th Street beneath Park Avenue would require tunneling in Manhattan and under the Harlem River as well as relocating, realining, and regrading both the New York Central and the New Haven tracks for a considerable distance in the Bronx at a cost variously estimated at from $200,000,000 to $350,000,000. The present proposal contemplates maintenance of the existing track system, which would be flanked in Manhattan by through and local express highways, with service roads. These highways would be treated and landscaped in such a manner as to eliminate any objectionable features to the adjacent proposed residential area, which will be sufficiently distant and screened from the railroad to provide, for most of the distance north of 96th Street, a prospect ever more inviting than that to the south. Including the acquisition of property on Park Avenue, making an artery 340 ft wide, an ornamental bridge across the Harlem River and a treated viaduct in the Bronx, the total estimated cost approximates $40,000,000, a major part of which would be recouped from increased land and building taxes. The further acquisition of an entire block on each side of Park Avenue, for park and residential purposes, has also been considered. The cost of such additional land would increase greatly the cost of the project, which is presently contemplated to be a self-liquidating toll highway.

It may be that the consummation of this proposal, as well as that of a Bronx Railroad terminal, will have to await the evolution of a new economic system wherein a nation possessed of a recognized need for improvements, a plentiful supply of man and machine power (much of which is still unemployed), an almost inexhaustible supply of material, an increasing fund of knowledge, and an engineering talent and ambition equal to that of any previous generation, will not be compelled to forego urgent projects for the betterment of mankind because of a stated deficiency of a medium of exchange presently termed money. The writer contemplates both a terminal and a highway as well as a new economic order in the not too distant future.

In one instance, an expressed conviction of the author may be open to question. The tremendous increase in land values in Westchester County, in the opinion of certain groups, is due rather to its parkway system than to railroad facilities. Almost identical claims are made by the proponents of rail and highway influences. Whatever the principal cause, there is ample room for the important rôle played by each. In fact, the cause of growth might be given a time index; the earlier expansion belonging almost exclusively to the railroad impetus and the more recent increase to the Westchester Parkway facilities.

It is eminently fitting that the paper should quote the distinguished engineer, then chief advisor to the Board of Estimate and Apportionment, who, in recommending that the city approve the plans submitted by the railroad, envisioned in his mind's eye the world's finest railway terminal. This vision long since has been realized. As an apostle and an authority on comprehensive municipal planning, the commendation of Nelson P. Lewis was, in this instance, a compliment as much to be valued as merited, and the tribute paid is but a suggestion of how an engineer can aid in giving due public acknowledgment to the talents and achievements of a fellow member of his profession.

It is not in the accomplishment of the Grand Central development alone that the vision and talent evidenced by this paper are apparent. The author's proposals for transportation facilities adequate for the needs and possibilities of the Metropolitan area, and his proposals for a New Jersey-Long Island Railroad freight tunnel and a connection between New Jersey and a point north of Manhattan, as well as certain other suggestions for improving Metropolitan services, land, water, and air, merit the consideration of planning agencies and public officials. It is from the dreams of imaginative men that mankind progresses. Colonel Wilgus is one who has witnessed in a goodly measure the realization of some of his dreams.

C. E. SMITH,[29] M. AM. SOC. C. E. (by letter).—Civil engineering is shown at its best in the conception, planning, and consummation of such a great engineering work as the Grand Central Terminal, described by Colonel Wilgus. The profession is fortunate indeed that Colonel Wilgus has given so much of his time and thought to gathering together, in such interesting fashion, what might otherwise have been rather dry statistics relating to one of the most important engineering projects of modern times. The paper is an important contribution to the history of the New York and Harlem, New York Central, and New York, New Haven and Hartford railroads, to the history of the City of New York, and to the history of civil engineering.

One important feature that was not mentioned in the paper relates to the ramp system which distinguishes the Grand Central Terminal from all other passenger terminals in the United States. Several years ago the writer had occasion to look up the background of that system and traced its inception back to Colonel Wilgus himself, who, in the absence of other evidence, is entitled to the entire credit for the inception and conception of the ramps which have relieved so many millions of passengers of the onerous task of climbing and descending stairs. That one feature alone is worthy of the greatest commendation.

There is one feature that has been developed since the construction of the Grand Central Terminal that would have added very materially to the convenience of passengers, particularly commuters, using that station, and that is the mezzanine between street level and track platform level. This has been developed quite successfully in the underground commuter terminal of the Pennsylvania Railroad at Broad Street, Philadelphia, Pa., and more recently in connection with the Sixth Avenue Subway of the City of New York. Such a mezzanine at Grand Central Terminal would have eliminated the single unsatisfactory feature that forces some passengers to walk twice the entire length of trains between the waiting room and the farthest cars to and from 45th and other streets. A mezzanine at Grand Central would have permitted passengers to enter and leave train platforms as far north as 45th Street without the necessity of passing the gates at about 43d Street. The convenience to commuters in the 45th Street district would have been great, but it is too late now to install the mezzanine except at an expense out of all proportion to the benefits that would follow.

Referring to the different systems of electric traction adopted by the New York Central and New Haven railroads, Colonel Wilgus states (see heading

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[29] Vice-Pres., N. Y. N. H. & H. R. R., New Haven, Conn.

"Grand Central Terminal Transformation—Formative Period from 1899 to 1907: Concept of All-Electric Service Throughout Suburban Region"):

> "The A.C.-D.C. struggle later was to assume the dignity of a 'celebrated case' comparable to the famous 'battle of the gages' in England in bygone years. It gathered force when the New York, New Haven and Hartford Railroad, owning trackage rights south of Woodlawn, belatedly adopted the views of Mr. Westinghouse and found it necessary to plan an electric locomotive that could operate on both alternating and direct currents."

The electrical engineers and management of the New Haven Railroad Company were bold indeed to venture into a new and relatively untried system of electrification—11,000-volt, 25-cycle, single-phase alternating current, which is carried directly to the locomotives and multiple-unit cars through the pantographs—particularly as it is also necessary to operate its trains on 600-volt direct current on the New York Central tracks between Woodlawn and Grand Central. However, continual research of the New Haven Railroad Company, in cooperation with the Westinghouse Electric and Manufacturing Company, has developed that system as the preferable system for long-distance, heavy-traction, railroad electrification. The New Haven-Westinghouse system has been adopted almost universally—with notable exceptions—throughout the world for heavy, long-distance electrification of railroads. The most recent convert to that system is the Pennsylvania Railroad Company in its electrification between New York, Washington, D. C., and Harrisburg, Pa., following the Great Northern, Norfolk and Western, Reading, and Virginian railway companies. If the New York Central Railroad Company were adopting a system of electrification today, it is doubtful if it would adopt its present system of third-rail, 600-volt direct current.

Colonel Wilgus' reference to a proposed new station at 149th Street and a loop just south of it is interesting indeed, particularly as interest has since been directed to that possibility by construction, underground at that point, of a two-level, rapid-transit station where express trains of the 7th Avenue and 4th Avenue subways are available; but the load on Grand Central has not yet reached the point where an auxiliary intermediate terminal at 149th Street would be justified.

Colonel Wilgus refers to the increase in assessed values in New York City and Westchester County, a considerable part of which, however, must be credited to the rapid-transit developments in New York City and to the highway development during the period. Just prior to the start of work on the present Grand Central Terminal, the city had placed in operation the first subway with loop terminal at City Hall, located on 4th Avenue south of Grand Central, on Broadway north of 42d Street, and running on 42d Street between 4th and 7th avenues. This was the first so-called "Z" subway, which was later enlarged into the east and west side subways by extending the 4th Avenue line north on Lexington Avenue, with branches in the Bronx, and by extending the Broadway subway south on 7th Avenue. Both lines were extended to Brooklyn. The former connecting link between Grand Central and Times Square now constitutes "the Shuttle." There is no doubt that this expansion of rapid transit, taken in connection with the development of the Grand Central Station, as well as the development of express highways, had a profound effect on property values in New York City and in Westchester County, notwithstanding which the Grand Central Terminal must be given credit for a large part of the increase in property values, both in and around Grand Central and in Westchester County.

Indeed, it may be said that the electrification of the New York Central and New Haven railroads, and the building up of Westchester County, may have had considerable to do with the decline in population in the Borough of Manhattan south of 42d Street, most of which, however, must be attributed to the development of rapid transit.

Just preceding the heading "Beneficial Results: Public Blessings," Colonel Wilgus shows a tabular comparison between passengers in and out of the Grand Central Terminal in 1930 compared with 1906. Under "Résumé and Conclusion: The Future" he states: "It would seem, therefore, that the time is not far distant when, as was done forty years ago (1899), thought must be given to the taking of measures for handling a further increase of traffic and for satisfying public demands." Table 7 shows a comparison between passengers handled in and out of Grand Central Terminal in 1930 and 1939. In the latter year, the New York, Westchester and Boston Railway did not operate, having suspended all operations on December 31, 1937. Also shown for comparison is 1936, the last year in which the New York, Westchester and Boston Railway operated in full. In that year, the New York, Westchester and Boston Railway handled 4,220,138 passengers to and from the 133d Street terminal at the Harlem River.

Although there was a tremendous increase in the number of passengers handled from 1906 to 1930, there was a decrease of slightly more than 20% from 50,643,000 in 1930 to 39,778,000 in 1939, even after including passengers using the Grand Central Terminal who formerly used the defunct New York, Westchester and Boston Railway. There are no indications at the present time of such an increase in traffic as would require any material enlargement of the capacity of the Grand Central Terminal and, needless to say, the job was so well done that public demands are still quite well satisfied.

TABLE 7.—PASSENGERS IN AND OUT OF GRAND CENTRAL TERMINAL (IN THOUSANDS)

Description	1930		1936		1939	
	Commuters	All	Commuters	All	Commuters	All
New York Central Railroad	26,264	33,765	15,824	21,437	16,481	22,470
New York, New Haven and Hartford Railroad	9,444	16,878	6,828	14,289	8,361	17,308
Total	....	50,643	....	35,726	....	39,778

Although Colonel Wilgus attributes the increase of passengers over a long period, from 1906 to 1930, to electrification, it is interesting to note that other

factors have a profound effect on the trend of passengers carried. This is illustrated by Fig. 15, showing number of commuter and other passengers handled in and out of the Grand Central Terminal by the New York, New Haven and Hartford Railroad each year from 1900 to 1915. It will be noted

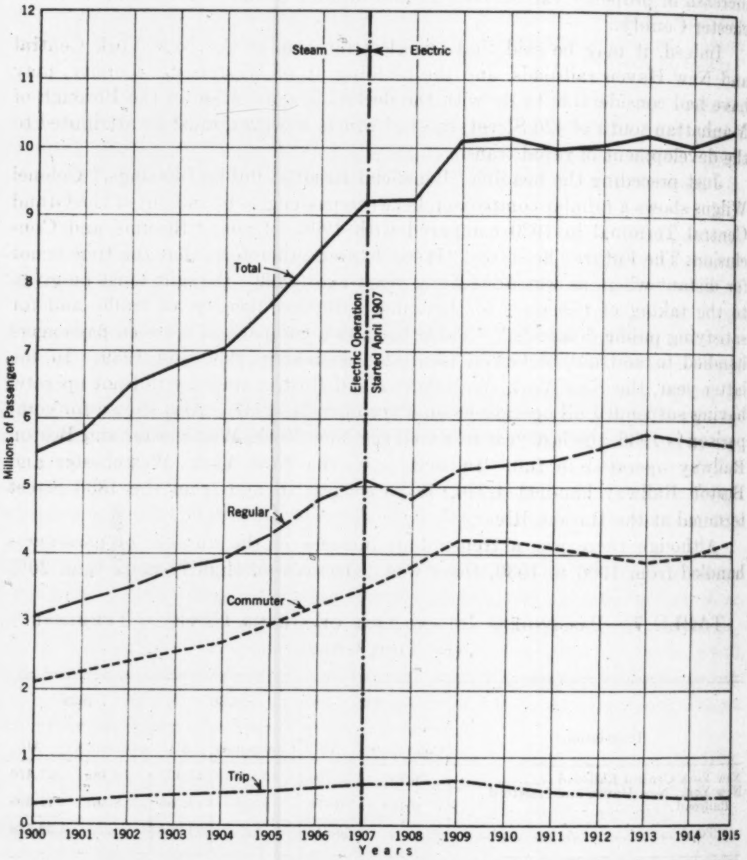

FIG. 15.—PASSENGERS ENTERING AND LEAVING NEW YORK CITY VIA THE NEW YORK, NEW HAVEN AND HARTFORD RAILROAD

that there was a rapid increase in the total number of passengers handled by steam locomotives from about 5,500,000 to slightly more than 9,000,000 in 1907, the year when steam operation was discontinued and electric operation started. This increased to approximately 10,000,000 in 1909. It remained at this level for seven years to and including 1915, indicating that electrification in and of itself was not sufficiently persuasive to continue the upward trend that

had been in effect seven years prior to electrification. Of course, the depression of 1907 and the ensuing stagnation of business was evident in preventing an increase of passengers until 1915, following which there was a considerable further increase stimulated by the first World War.

Harold M. Lewis,[30] M. Am. Soc. C. E. (by letter).—The transformation wrought in the vicinity of the Grand Central Terminal in the City of New York, as well as the incidental changes that have taken place in the suburban areas which it serves, have been described forcefully by Colonel Wilgus. These justify his calling the project "a successful adventure in civic planning." This discussion is limited to the general relationship of the project to the plan of the city and region within which it lies. From this point of view, it is of interest to consider what further improvements may be made in the approaches to the terminal, in either Manhattan or the Bronx, and how these might bring about improved development of adjacent territory. In addition, one would like to know to what extent other terminal areas might be transformed as a result of a shift from steam to electric operation.

The following two improvements to the approaches would have a far-reaching effect on abutting properties (see Fig. 16): (1) An elimination of all or part of the viaduct and elevated structure in Manhattan north of 96th Street; and (2) the development of an important sub-terminal in the vicinity of Mott Haven in the Bronx. Each of these has been mentioned by Colonel Wilgus.

The writer appreciates that any change in the New York Central Railroad tracks in Park Avenue would present a very difficult and costly problem for the railroad company, particularly if it should involve any change in the tracks now in the tunnel south of 96th Street. The study of the Regional Plan Association, Inc., referred to by Colonel Wilgus, proposed to leave these latter tracks, as well as those on the existing viaduct as far as 104th Street, unchanged in both grade and alinement, and involved track changes only north of 104th Street.

From that point, the tracks would descend at an easy grade into a tunnel at 116th Street, which would be facilitated by the fact that the grade of Park Avenue rises suddenly at this point. Between 116th and 122d streets this tunnel would swing through the blocks westward into Madison Avenue, which it would follow north of that point to a tunnel crossing of the Harlem River. In the Bronx, connections would be made to the Hudson River and Putnam divisions near McCombs Dam Bridge, to the Harlem Division near 163d Street and to the New York Central storage yard near 161st Street. The maximum grades involved on the main lines were 1.5% and about 1.7% on the connection to the storage yard.

The three main objectives of this plan were: (1) To provide for an improvement of Upper Park Avenue and an improved highway connection between Manhattan and the Bronx; (2) to provide a connection with a possible future suburban transit line in Madison Avenue, as proposed many years ago by the Westchester County Transit Commission; and (3) to eliminate interference

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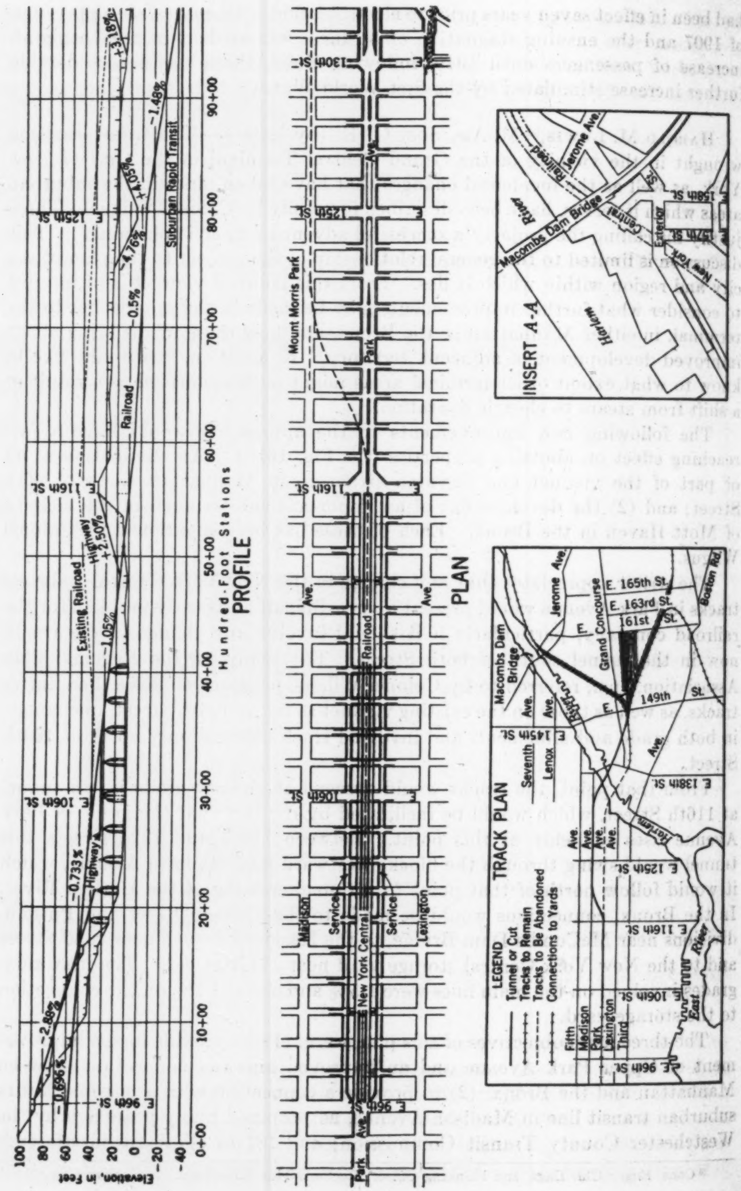
[30] Cons. Engr.; Chf. Engr. and Planning Officer, Regional Plan Association, Inc., New York, N. Y.


Elevation, in Feet
100
80
60
40
20
0
−20
−40
0+00
10+00
20+00
30+00
40+00
50+00
60+00
70+00
80+00
90+00
Hundred-Foot Stations
PROFILE
E. 96th St.
E. 106th St.
E. 116th St.
E. 125th St.
−0.696%
−2.88%
−0.733%
Highway
−1.05%
Existing Railroad
Highway
+2.50%
Railroad
−0.5%
−4.76%
+4.05%
−1.48%
+3.18%
Suburban Rapid Transit
PLAN
Park Ave.
Madison Ave.
Service
New York Central Railroad
Service
Lexington Ave.
E. 96th St.
E. 106th St.
E. 116th St.
E. 125th St.
E. 130th St.
Mount Morris Park
TRACK PLAN
LEGEND
Tunnel or Cut
Tracks to Remain
Tracks to Be Abandoned
Connections to Yards
Fifth
Madison
Park
Lexington
Third
Ave.
E. 96th St.
E. 106th St.
E. 116th St.
E. 125th St.
E. 138th St.
E. 145th St.
Seventh Ave.
Lenox Ave.
Harlem River
East River
Macombs Dam Bridge
Jerome Ave.
Grand Concourse
E. 149th St.
161st St.
E. 163rd St.
E. 165th St.
Boston Rd.
INSERT A-A
Harlem River
Macombs Dam Bridge
Central Railroad
New York
Jerome Ave.
E. 157th St.
E. 158th St.

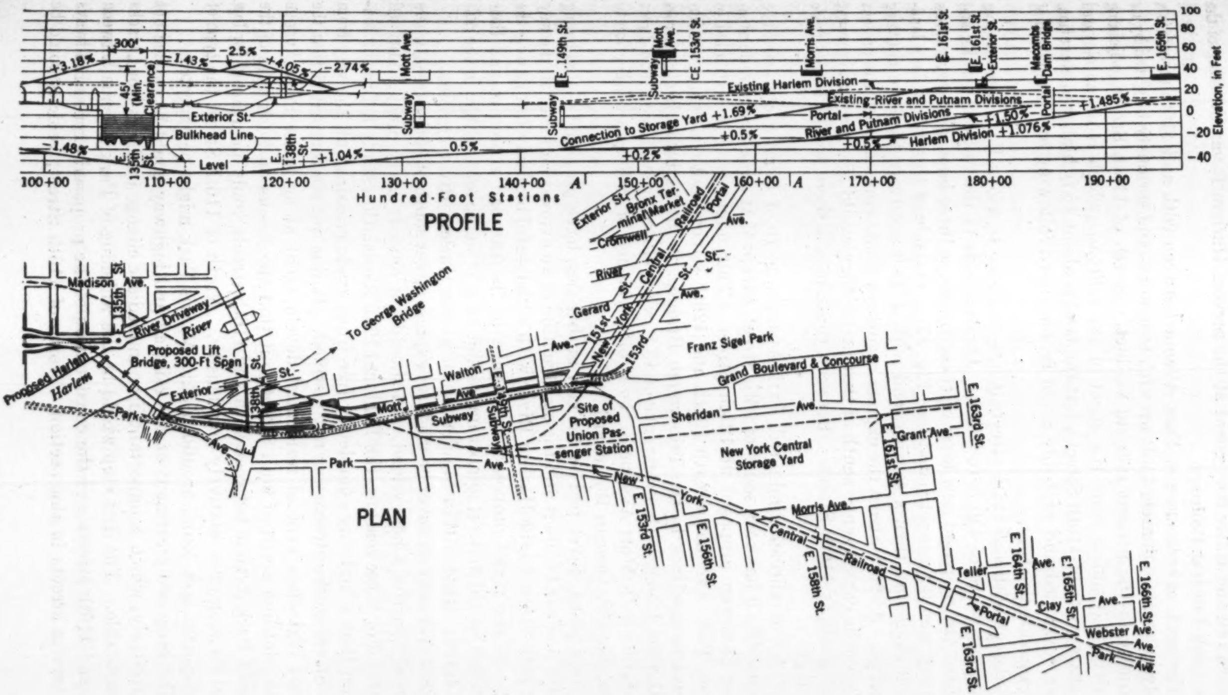

100
80
60
40
20
0
−20
−40
Elevation, in Feet
+3.18%
300'
−1.43%
−2.5%
+4.05%
−2.74%
45' (Min. Clearance)
Exterior St.
Bulkhead Line
−1.48%
E. 135th St.
Level
E. 138th St.
+1.04%
0.5%
Mott Ave.
Subway
E. 149th St.
Subway
Subway Mott Ave.
CE. 153rd St.
Morris Ave.
E. 161st St.
E. 161st St.
Exterior St.
Macombs Dam Bridge
E. 165th St.
Existing Harlem Division
Existing River and Putnam Divisions
Portal
Connection to Storage Yard +1.69%
+0.5%
Portal
River and Putnam Divisions
+1.50%
+1.485%
+0.2%
+0.5%
Harlem Division +1.076%
100+00
110+00
120+00
130+00
140+00
A
150+00
160+00
A
170+00
180+00
190+00
Hundred-Foot Stations
PROFILE
Madison Ave.
135th St.
River Driveway
River
Harlem
Proposed Harlem
Proposed Lift Bridge, 300-Ft Span
To George Washington Bridge
Exterior
138th St.
St.
Park Ave.
E.
Walton Ave.
Mott Ave.
Subway
E. 149th St.
Subway
Park Ave.
Exterior St.
Bronx Terminal Market
Cromwell
River St.
Gerard Ave.
151st St.
New York Central Railroad
Portal
Ave.
St.
153rd
Franz Sigel Park
Grand Boulevard & Concourse
Site of Proposed Union Passenger Station
Sheridan Ave.
E. 161st St.
Grant Ave.
E. 163rd St.
New York Central Storage Yard
New York Central Railroad
E. 153rd St.
E. 156th St.
Morris Ave.
E. 158th St.
Teller Ave.
E. 164th St.
Portal
Clay Ave.
E. 165th St.
E. 163rd St.
Webster Ave.
Park Ave.
E. 166th St.
PLAN

FIG. 16.—STUDY FOR IMPROVEMENT OF PARK AVENUE, NEW YORK, N. Y.

between rail and water movement at the present Harlem River Crossing of the New York Central Railroad.

The area on both sides of Park Avenue between 96th and 116th streets is a low-lying section, densely built up with tenements and seriously blighted by its proximity to the present railroad viaduct. North of 116th Street, adjoining property still suffers from the effect of the railroad (which is on an elevated structure north of 110th Street), but the land is about 15 ft higher in elevation, better developed and probably could be reclaimed following an elimination of the railroad structure.

Between 96th and 116th streets it was proposed to widen Park Avenue from its present width of 140 ft to 300 ft. On each side of the railroad tracks, and at about the proposed new level of these tracks, a three-lane, one-way express roadway would be constructed (see Fig. 17). Outside of this would be a landscaped slope and then a service road with a 30-ft roadway to serve abutting property. At 96th Street the express roadways would connect with the present roadways in Park Avenue south of that point; they would also meet the present street grade at 116th Street. Between those points there would be no grade crossings.

With the railroad eliminated, Park Avenue, north of 116th Street, could be developed as it has been south of 96th Street, except that a highway grade separation has been proposed at 125th Street. This might well be extended to include Park Avenue beneath 124th and 126th streets, both of which are important connections to the Triborough Bridge. A lift highway bridge across the Harlem River, with a clearance of 45 ft when closed, would provide a direct connection with Mott Avenue in the Bronx, widened in 1940 as a connection to the William F. Deegan Boulevard.

Other plans, developed by the city authorities, have proposed the widening of Park Avenue to 300 ft and the construction of an express highway all the way from 96th Street to the Harlem River with a high-level highway bridge over the river. These plans proposed no change in the railroad structure and their execution would make it almost impossible to ever make any change in the railroad tracks either within Manhattan or at the Harlem River crossing.

Park Avenue can never be made an express street south of 96th Street, where it already carries a heavy load, and should not have to carry much additional traffic. For these reasons, the Regional Plan Association study led to the conclusion that a highway completely free from grade crossings all the way from 96th Street to the Bronx was not justified. It was not contemplated that the present high-class type of residential development along lower Park Avenue would continue north of 96th Street, but that the blocks on both sides of the widened Park Avenue between 96th and 116th streets could be utilized for low rental housing at a relatively low density. North of 116th Street the type of development now found around Mount Morris Park might be expected.

The proposed treatment of the railroad and highway could be carried out progressively, which is important if no immediate change in the railroad tracks is practicable. The first step would be the widening of Park Avenue between 96th and 116th streets and the construction of the proposed express roadways and service streets in this section. North of 116th Street, vehicles could be accommodated partly on the surface of Park Avenue, beneath and alongside the railroad structure, and partly by diversion to Madison and Third avenues.

The railroad plan north of 116th Street could be carried out later without any interference with existing tracks and services, as both the line in Madison Avenue and the proposed Harlem River tunnels are far removed from any existing railroad structure. The railroad changes between 104th and 116th streets could be facilitated by diverting the existing tracks to temporary trestles constructed in the landscaped areas between the express roadways and service streets, placing two tracks on each side. Some temporary changes would have to be made in the express roadways between 103d and 106th streets, reducing temporarily the clearance where 104th and 105th streets would pass beneath.

Following this temporary relocation of the tracks, the existing viaduct could be rebuilt at the proposed new grade, a connection made with the new tunnels north of 116th Street and the railroad viaduct removed north of that point.

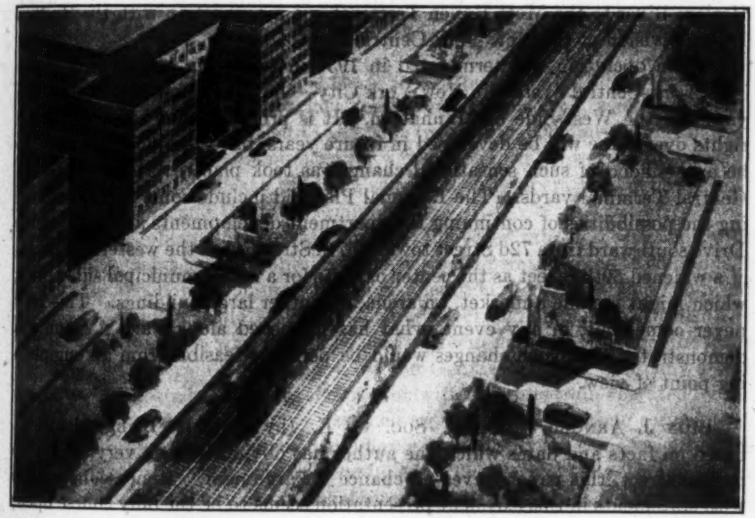


Fig. 17.—Proposed Treatment of Park Avenue, Manhattan, Between 96th and 116th Streets (A Widened Right-of-Way Would Permit the Construction of Express Highways On Each Side of the Railroad Right-of-Way)

A final step would be the construction of the proposed highway bridge over the Harlem River to connect Park Avenue with the Grand Boulevard and Concourse.

The Regional Plan contemplated the development of a series of important sub-centers within New York City as subsidiaries of the main business center, which it was expected would remain in midtown Manhattan. There is already a center of this type in Harlem, focused about 125th Street, but it was expected that another important business district would arise in the Mott Haven section of the Bronx. With that in mind, parts of the proposed regional highway and rail systems were shown as focused in this area where a union railroad passenger terminal was proposed. Colonel Wilgus has referred to such a terminal as a part of the plans of the New York Central Company that "has not been executed in full."

The study, by the Regional Plan Association, for the Park Avenue improvement showed this terminal just north of 149th Street and east of the new Bronx Post Office. It would be located beneath the present yard level and its platforms would serve eight tracks, connected through a mezzanine. Convenient connection could be provided with both the East Side and West Side subway systems in Manhattan.

From such a terminal, rapid bus connections could be provided to the Borough of Queens over the Triborough Bridge, easily reached by the William F. Deegan Boulevard. The Regional Plan also contemplated that when a suburban transit line from New Jersey is supplied over the George Washington Bridge, as provided for in the bridge design, this might be extended over the Harlem River to the Mott Haven Terminal, via a connection with the Hudson River Division of the New York Central Railroad.

Steam operation was terminated in 1931 in two other large yards of the New York Central System in New York City—the 30th Street and 60th Street yards on the West Side of Manhattan. It is probable that some of the air rights over these will be developed in future years, but there does not seem to be a likelihood of such sensational changes as took place over the old Grand Central Terminal yards. The Regional Plan did include some pictures showing the possibilities of continuing the apartment developments along Riverside Drive southward from 72d Street toward 57th Street, with the western terminus of a widened 59th Street as the center of a site for a major municipal sub-center, which might include a market, an arena, and other large buildings. This may never come, but, in any event, what has happened along Park Avenue has demonstrated that such changes would be perfectly feasible from an engineering point of view.

BION J. ARNOLD,[31] M. AM. SOC. C. E. (by letter).—Being historical, based on facts and dates which the author has obviously been very careful to make correct, this paper leaves no chance for argument. Hence concurrence with statements in this excellent presentation seems to be the only course open; and especially is this so when one has had a part in compiling some of the data upon which final action was based.

As Colonel Wilgus states, it was the writer's lot to be called in August, 1901 (see heading "Grand Central Terminal Transformation—Formative Period from 1899 to 1907: Initial Move for Electrification"), "to study the feasibility of handling heavy through trains by electricity between Mott Haven and the terminal"—thus he was the first outside consultant upon whom the responsibility of securing definite conclusions and recommendations regarding the electrification was placed. Later he was a member of the Electric Traction Commission, which carried the responsibility of the actual designing and construction work. As he was paid a fixed fee for his investigation and report, he was at liberty to adopt any method he thought advisable to secure the necessary information upon which to base conclusions.

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[31] Cons. Engr., Chicago, Ill. (Col., Air Corps, Inactive Reserve, U. S. Army).

At that time, although there had been a number of railway electrifications of mixed types, there was no electrification approaching the magnitude of this one and no data that the writer could find, which could guide him in determining the amount of power required to propel heavy trains under conditions such as this installation involved. Colonel Wilgus, then chief engineer, made available all the facilities owned by the railroad company that were needed.

At that time there were few dynamometer cars in existence in the United States; but one car was owned jointly by the Illinois Central Railroad Company and the University of Illinois, at Urbana, Ill. In order to be certain that the car would be properly operated, the writer employed Edward C. Schmidt, Assoc. M. Am. Soc. C. E., then professor of railway mechanical engineering at the University of Illinois, and his staff. This car was coupled between the locomotive and the train on one of the heavy passenger trains operated between Chicago and New York and then used in the same way to measure the draw-bar pull of many trains running into and out of the Grand Central Terminal in New York, requiring several weeks.

From the records thus secured the draw-bar pull required to accelerate and propel each class of train at various speeds was obtained and when reduced to a horsepower basis and applied to all trains (more than 600 daily) that ran over the division, the total maximum power was determined, as well as the average power required to duplicate the train service then in operation between Mott Haven and the Grand Central Terminal.

The next problem was to ascertain what increased rate of acceleration with the different trains could be obtained by electrical operation and the consequent increase in the power required for such operation. To meet this condition arrangements were made with the General Electric Company to furnish the use of two motor cars, which could be used in a series of tests to propel the train furnished by the railroad company on the General Electric Company's test tracks at Schenectady, N. Y., in comparison with the same train operated by the most powerful steam locomotive designed especially for suburban service that the railroad company then owned and having practically the same weight on its drivers as the two motor cars had combined. The electric tests were made on the General Electric test track, located on the berm bank of the old canal at Schenectady; but the curve leading from the New York Central tracks into this track was too sharp for the locomotive to go around it, so that the tests of the steam locomotive and the trains were made on the west-bound freight track of the main line west of Schenectady, proper adjustments being made for grade, curvature, and wind resistance on the different runs. These tests, and the necessary mathematical calculations and deductions from them to reach conclusions, involved a vast amount of work by many men.[32]

In determining the form and best method of producing and applying the electricity necessary to propel the trains, twelve different types and locations of power stations and methods of distribution were outlined and their relative efficiencies estimated. In general, the results of these tests and analyses showed, and the writer's recommendations were to the effect, that the electrification of the terminal between Mott Haven and the Grand Central Station was feasible. The saving in operation effected thereby was only slightly in favor of electrification when the fixed charges, made necessary by the installation, were taken into account. Therefore the installation could not be justified because of these economies of operation alone; but it could be justified on other grounds such as safety, increased capacity, etc., or by the extension of the electrified zone. This conclusion has been thoroughly demonstrated as sound by the results of many years of successful and profitable operation, as testified by Colonel Wilgus.

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[32] "Method of Ascertaining by Means of a Dynamometer Car the Power Required to Operate the Trains of the New York Central & Hudson River Railroad Between Mott Haven Junction and Grand Central Station, and the Relative Cost of Operation by Steam and Electricity," by Bion J. Arnold, *Transactions*, Am. Inst. of Electrical Engrs., Vol. XIX, 1903, p. 865; also "Comparative Acceleration Tests with Steam Locomotive and Electric Motor Cars," by W. B. Potter and Bion J. Arnold, *loc. cit.*, p. 833.

When engaged on the work of this terminal the writer had the good fortune to occupy joint offices with Charles A. Reed, the constructive genius of Reed and Stem, the architects who, working as a firm with the railroad officials, prepared the first plan (see Fig. 9) showing a twenty story station with its accompanying court of honor. This plan of utilizing "air rights" so impressed the writer that when later acting as consultant for the City of Chicago, Ill., on railway terminals, after months of strenuous argument, he saw his own ideas in this respect prevail. As a result the vast area then owned by the railroads contiguous to, and west of, the Chicago River began to be dotted with "air right" structures. Prominent among these structures are the new U. S. Post Office, the Pennsylvania Railroad Freight Station, and the Chicago Daily News Building, forming the nucleus, if electrification ever comes, of another Grand Central Development of "air right" structures.

Later the same "air right" ideas prevailed in the minds of those who developed the railway station at Cleveland, Ohio, and the writer is of the opinion that the surplus strength in the footings and heavy columns which Mr. Reed and Colonel Wilgus succeeded in embodying in the present Grand Central Station Building in New York (but now lying dormant) will yet be utilized by the extension of the station building upward. Thus another step will be completed, as conceived in the original plan, which like several other steps mentioned in the paper, although delayed, have shown their merit and been completed since.

The author of the paper is to be commended for his thoroughness in compiling the vast amount of historical and engineering data, and the Society is to be congratulated upon having this matter spread upon its records in such satisfactory sequence and form.

JAY DOWNER,[33] M. AM. SOC. C. E. (by letter).—All those who have been concerned with the development of Westchester County are enthusiastic in their appreciation of the important influence of the Grand Central Terminal on the growth and character of the suburban territory to the north of New York City. It has been recognized that in all respects this terminal is the best in the country, and its convenience as compared with other large stations has been particularly appreciated. Colonel Wilgus has presented a comprehensive and readable account of the inception and execution of this great undertaking, which eventuated from the foresight and ability displayed by him and his associates.

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[33] Cons. Engr., New York, N. Y.

Without in any way detracting from the importance of this improvement, some exception may be taken to the assumption of full credit for the increase in assessed valuations in Weschester County, which rose from about $172,000,000 in 1900 to $1,828,000,000 in 1932. During the construction of the Bronx River Parkway, valuations rose from $383,000,000 in 1913 to $786,000,000 in 1923, after which, and during the development of the more comprehensive Westchester County Park System, the rise was accelerated to a high level of $1,828,715,000 in 1932. During these nine years the county, with some aid from the state, had expended upwards of $77,000,000 for its park and parkway system, which was in addition to the earlier Bronx River Improvement costing about $17,000,000. In addition, trunk sewer systems and a county highway system helped to develop Westchester's suburban area.

Therefore, much as one may appreciate the fundamental importance and basic value of the Grand Central Terminal and the electrification of the three railroads leading through the county, one feels justified in asking the railroads to share with the far-flung parkway, trunk sewer, and highway systems some of the credit for the increase in values that were notably accelerated during the construction of these county improvements. At one period those who were instrumental in promoting the County Park System claimed credit for 75% of this increase. In Colonel Wilgus' paper it is assumed that the railroads may be credited with 100%. Each major improvement was responsible in greater or lesser degree for the phenomenal rise in values, and it would be impossible to apportion the credit accurately. Perhaps there is credit enough for all. Certainly the Grand Central Terminal and the electrification of the railroads contributed magnificently to the amenities of life in Westchester.

William J. Wilgus,[34] Hon. M. Am. Soc. C. E. (by letter).—In this closure to his four-dimensional paper, in which the "element" of time has played a part, the writer first would express his deep sense of appreciation to the Society in general for having given it a place in its publications, and in particular to those of his fellow members whose scholarly contributions have given to it life and color. Thus has the product of the spirit, brain and brawn of many distinguished men and their aides in engineering, architecture, contracting, law, finance, operation, and administration been given "its day in court."

Both Mr. Hammond and Colonel Arnold point to the example set by the Grand Central Terminal in the utilization of "air rights" under somewhat similar circumstances at Chicago and, as Colonel Arnold states, at Cleveland. In this connection it should be remarked that the first plan of the Grand Central Terminal showing a lofty station building and adjoining hotel was not that of Mr. Reed, as thought by Colonel Arnold, but that of Samuel Huckel, Jr., who prepared such a study to illustrate the writer's plan for the improvement based on the "air rights" idea submitted to Mr. Newman, President of the railroad, on December 22, 1902. Mr. Reed's brilliant conception of the Court of Honor and circumferential driveways came some months later, and brought to his firm its engagement as architects for the station building and later for the hotel. This has been explained and illustrated in some detail in the complete manuscript.[21]

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[34] Weathersfield, Ascutney P.O., Vt.

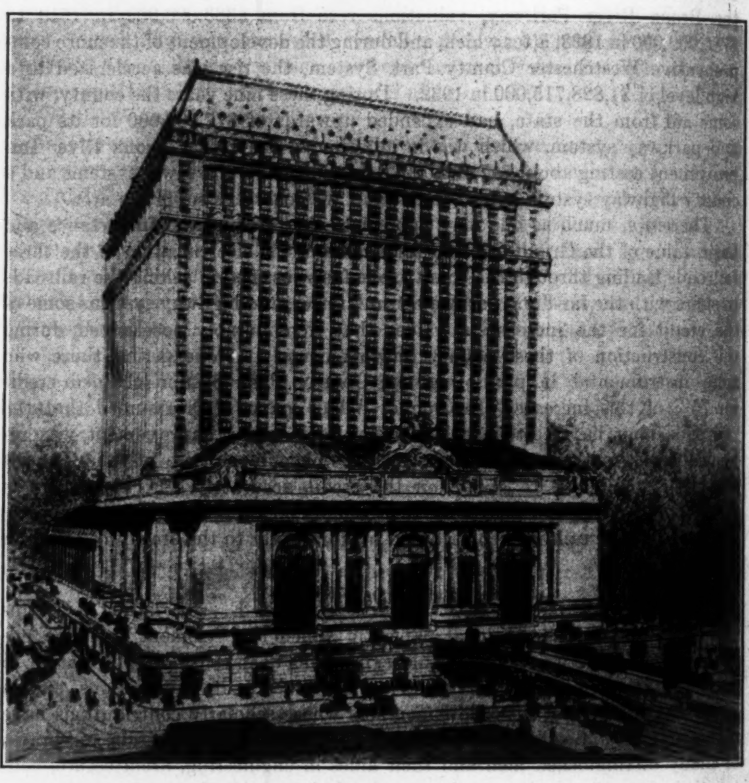

FIG. 18.—FUTURE GRAND CENTRAL TERMINAL (IF AND WHEN A HIGH BUILDING IS ADDED SURROUNDING THE CONCOURSE)

It is indeed to be regretted, as expressed by Major Hallihan and Mr. Sheridan, that the Court of Honor in its original form was eliminated from the final plans. Had it been permitted to remain, the community would have profited from a separation of street grades all the way north to 49th Street, and from the obvious advantages of a plaza of majestic proportions such as are found in similar situations in Europe. It is not too much to believe that give-and-take negotiations between the city authorities and the railroad might have resulted in a just division of the financial burden and thus have made possible the retention of the Court-of-Honor plan.

Mr. Smith refers to the mezzanine which, if installed at the Grand Central Terminal between street level and track platform level, would have added very materially to the convenience of passengers. Undoubtedly this is true were it practicable to bring it about, but unfortunately this would have required the raising of street levels and the lowering of track levels which were not permissible.

Mr. Hammond refers to the wisdom of providing separate platforms for the handling of passengers and baggage. Unfortunately, as in the case of the mezzanine, this was not possible at the Grand Central Terminal because of physical limitations. Studies were made for baggage platforms equipped with mechanical conveyers between the running tracks; but this required the objectionable transfer of the building columns to the middle of the passenger platforms and the lessening in number or narrowing of the latter within the inexpansible width of the terminal area. It was a matter of great regret to the writer that separate means for handling baggage could not be devised without an inadmissible sacrifice of passenger platform space. Mr. Hammond also mentions the happy effect of sunlight streaming through gable windows upon crowds of passengers assembled in the station concourse. It is to be expected that if and when the lofty building is added to the structure, as illustrated in Fig. 18, pains will be taken not to destroy this feature.

Mr. Meehan points to dangers incident to the two right-angled turns in each direction where the circumferential elevated driveways come together at 42d Street. It would be wise indeed to give earnest thought to his suggested remedy. When the structure was built the planners did not sufficiently visualize the speed and volume of street traffic that were to come in later years.

In respect to the change of motive power at the terminal and on its approaches, Mr. Smith considers that "If the New York Central Railroad were adopting a system of electrification today, it is doubtful if it would adopt its present system of third-rail, 600-volt direct current." Unquestionably his company, the New Haven, and the other roads he mentions, have, in the high tension alternating current system, an admirable one for their conditions as proved by the passage of time. His road is indeed to be congratulated on having led the way; but conditions on the New York Central were not of the same order. As is said by Colonel Arnold, at the time when the problem of changing its motive power from steam to electricity arose, there was "no electrification approaching the magnitude of this one." In the words of Mr. Hill, "electric traction was still in its formative stage." On top of this departure from precedent went the need, referred to by Messrs. Lavis, Hill, and Ripley, of keeping traffic moving—without delay or serious inconvenience to the public—at the terminal itself, where hundreds of steam-operated trains daily had to be kept moving "on time," coupled with innumerable horizontal and vertical changes of position of the tracks during two-level, underneath and overhead construction. This condition, if for no other reason, barred the use of exposed high tension wires supported from poles, as did the narrow clearance of only one inch above the top of steam locomotive smoke-stacks in the Park

Avenue tunnel bar the use of high tension working conductors suspended from its roof. Apart from these physical obstacles there was the legal obstacle to the erection of pole-supported wires on the Park Avenue Viaduct and elsewhere within the city limits. Along with these prohibitory conditions went the state of the art which led the members of the Electric Traction Commission unanimously to decide that in recommending so revolutionary a change of motive power affecting the safety and reliability of a vast number of through and suburban train movements, they should adhere to what was then deemed to be least experimental. The adopted system, so far as the writer knows, has worked successfully since the running of the first electric train on September 30, 1906. Were the work to be done over again, and under present-day conditions, it is very possible, and even probable, that the New Haven method would receive serious consideration. The physical obstacles to its adoption in the first decade of the century have disappeared and perhaps those of a legal nature could be removed.

Mr. Smith gives some very interesting data showing the let-up in the rate of increase in his company's number of passengers entering and leaving New York City during and immediately following the depression from 1907 to 1915. With continued steam operation after 1907 it would have been impossible to have won much, if any, increase. The old Grand Central Station yard had reached the limit of its capacity, and the terrors of the smoke- and gas-infested Park Avenue tunnel had their deadening effect on further growth of Westchester County suburban communities. That the electrification, in the long run, did promote a marked upward trend in the traffic of both roads using the terminal is shown by the tabulated data in the paper.

As to planning for coming years, Mr. Hill very wisely asks who will say what the future holds? But is not all life a gamble? No one knows what the morrow may bring forth; and yet one may prophesy as best he can from day to day in preparation for what he believes may occur. Mr. Smith points to the decrease of some 20% in number of passengers handled at the terminal between 1930 and 1939 as indicating no need for material enlargement in its capacity and none for an auxiliary terminal at 149th Street in the Bronx (Mott Haven). As an auxiliary the writer agrees that, no doubt, a Mott Haven terminal is unnecessary; but with Mr. Sheridan and Mr. Lewis he considers it to be highly necessary to the community it would serve numbering a million and a half people. It would not only there afford a meeting place for rail, highway, and air carriers to and from nearby and distant regions, but also, in the improvement of the great Mott Haven open yard (now a deterrent to the progress of that community), it would do for the Bronx what the Grand Central Terminal has done for mid-Manhattan.

Mr. Sheridan and Mr. Lewis offer differing suggestions for an improved Park Avenue north of 96th Street and extending to the site of the proposed Bronx Terminal at 149th Street and beyond. It would seem clearly evident that the one of these to be approved by public authority, including the "grand-centralizing" of the Mott Haven yard, should have its details developed in full for immediate use when the United States again will be forced to make a frantic search, as in 1933, for worthy projects on which to engage its idle men, machinery, and materials otherwise going to waste.

Messrs. Downer and Lavis question if the building up of Westchester County can be attributed to the terminal improvement and electrification of its approaches to the extent indicated in the paper. In this they are joined by Mr. Sheridan, and as well by Mr. Smith, in respect of both the Grand Central Zone and the region north of New York City. The writer now realizes that he did not make sufficiently clear that, in designating the improvements in question as the "primary cause" of the increase in assessments, the "impulse" that started things going, the "spark" that started a train of events, he failed to dwell as he should on what followed, such as the erection of magnificent structures between Madison and Lexington avenues from 42d Street to 96th Street, and the initiation and creation of the remarkable parkway system and accompanying high-class residential development in Westchester County. The thought he intended to convey was that the beginning in these respects—and there always has to be a first cause—laid in the enterprise he has attempted to describe, very much as the child is looked upon as the father of the man, the acorn as the oak's beginning, and the Erie Canal as the greatest single factor in bringing to the State and City of New York their preeminence in the nation during the first half of the nineteenth century. If in this he is deemed to have gone too far, it is, he hopes, not to be taken as a lack of attachment to the cause of truth.

The writer intended to establish the claim that the transformation in question eliminated deterrents to developments that followed. The limitations of the Grand Central Station on further train service, the dreadful dangers and discomforts of the Park Avenue tunnel, the dirt and noise of steam-operated trains, the hazards of grade crossings and the occupancy of city streets in such important communities as Mount Vernon, White Plains, and Yonkers—all these disappeared with the changes made financially feasible by the utilization of air rights which, until then, had been fallow.

Then came the realizations that Park Avenue could be made a fashionable thoroughfare of the first order and Westchester County a fruitful field for parks and parkways having no peer.

In conclusion the writer would indulge himself in a renewal of his thanks to his fellow members who have so generously and enlighteningly given of themselves in the discussion, and also to those who will have paid him the compliment of reading through this record to the end.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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Paper No. 2120

THEORY OF ELASTIC STABILITY APPLIED TO STRUCTURAL DESIGN

By Leon S. Moisseiff,[1] and Frederick Lienhard,[2]
Members, Am. Soc. C. E.

With Discussion by Messrs. Louis Balog, Joseph S. Newell, Harold D. Hussey, H. N. Hill, F. H. Frankland, and Leon S. Moisseiff and Frederick Lienhard.

Synopsis

The theory of elastic stability is a study of the fundamental laws that govern the behavior of metals in compression and the application of the knowledge derived from such study to the design of structures. A comprehensive understanding of this behavior will enable the engineering profession to establish rules of design in accordance with the fundamental laws and thereby to construct more dependable as well as more economical structures. A greater freedom in the application of metals will result therefrom.

The elements and shapes in which metals are used for structural members have been studied and tested individually, as well as in combined forms. Tests of members subjected to compression have shown that the member as a whole will fail by flexure as a column, or its component parts will eventually wrinkle into waves. The stress at which these waves become visible depends on the material, the proportions of the elements, and the structural composition of the member. Elements that have wrinkled into visible waves can no longer sustain their proportionate share of the load and a small increase will cause failure.

It has been observed that for certain proportions of structural elements the waves become pronounced when the load approaches the yield point of the material. To obtain a "rough-and-ready" rule for the stability of the elements, the complex behavior of the material, heretofore, has been expressed conveniently by a simplified relation of width to thickness of plates or shapes. Such a relation forms the basis for the rules of design of most structural speci-

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Note.—Published in January, 1940, *Proceedings*.

[1] Cons. Engr., New York, N. Y.

[2] Engr., Leon S. Moisseiff, New York, N. Y.

fications. The very simplicity of the rule, however, confines the application to narrow limits and hampers the design and its economy.

Columns are generally composed of plates and shapes. To develop higher unit stresses in columns, low slenderness ratios are required and at the same time the designer is forced to adhere to specified ratios of plate width to plate thickness. The possibility of designing economical compression members is limited thereby. Such column design is exemplified by the towers of the Golden Gate Bridge in California for which carbon steel was used for the greater part to meet the buckling requirements of the web plates. The same limitation confronts the engineer in the design of plate girders to such a degree that long plate girders so proportioned become uneconomical.

It is intended, in this paper, to establish the buckling resistance of plates or webs reinforced by stiffeners and to develop methods of design which will enable the engineer to utilize the higher buckling resistance of the reinforced plates to their full capacity. This will enlarge the possibilities of forms of design and will effect considerable economy in the resulting structures.

It is not the intention to change any of the established rules but rather to propose additional requirements within the framework of the existing specifications. The rules thus established will become necessarily more complicated, but the additional work will be well compensated by the results.

In this paper the behavior of structural carbon steel, silicon steel, and an aluminum alloy of the duralumin type[3] have been studied, rules of design established, and tables and diagrams prepared for them. Following the same reasoning, the paper can be extended to any other metals by the use of the same formulas.

For the application of the theory of elastic stability to compression or bending of plates reinforced by stiffeners, detailed studies were made and simple rules for practical design were established. These rules can be extended to combined compression and bending. Additional diagrams and tables would have to be made for various values of the stability coefficient between the limits of 4 to 24. Such tables, however, would apply to special structures only, as stiffening girders of suspension bridges; these structures will deserve special studies.

Tests on the behavior of steel reinforced by stiffeners have been made, especially in Europe, to verify theoretical investigations in this line. They have been limited to some phases of the problem. To assure a wider application of the type of structures of which the paper treats, comprehensive tests are needed which should verify the theoretical considerations and indicate the best methods of fabrication. The combined results of theory and tests will open a wide field for structural design.

CHAPTER 1.—ELASTIC STABILITY

To acquire a full knowledge of the buckling stability of plates and shapes, a comprehensive and thorough study of the behavior of the metals in compression is required. Such knowledge will lead to conclusions which should be applicable to the design of structures. An approach to an understanding of the complex behavior of metals in compression has been made in this paper through a detailed study of the elastic stability of plates.

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[3] Known commercially as Aluminum 27S-T.

General Expression for Elastic Stability

The behavior of plates subject to compression, shear, or a combination of the two, can be expressed in the general form:

$$\sigma_{cr} = \tau_{cr} = k \frac{\pi^2 E \zeta}{12 \left[1 - \left(\frac{1}{m} \right)^2 \right]} \left(\frac{t}{d} \right)^2 \ldots\ldots\ldots\ldots\ldots (1)$$

in which: t = thickness of plate; d = width of plate; $\frac{1}{m}$ = Poisson's ratio; σ_{cr} = critical compressive stress at which buckling will occur; τ_{cr} = critical shear stress at which buckling will occur; E = modulus of elasticity in compression; k = stability coefficient depending on the dimensions and design of the plate, its end conditions, and kind of stress to which the plate is subjected; and ζ = modulus factor, a value by which Young's modulus of elasticity should be multiplied when the stress exceeds the elastic range.

The critical stress can be computed readily if an expression is established for the stability coefficient k and a numerical value for the modulus factor ζ. Eq. 1 also holds for the critical shear stress τ_{cr}, if the value of the modulus of elasticity in shear is accounted for in the coefficient k. This has been done in the paper.

Expressions for Stability Coefficient, k

Plates Subject to Direct Stresses.—The direct stresses to which a plate is subjected may be expressed (41*a*)[4] in the form:

$$\sigma_y = \sigma_0 \left(1 - \alpha \frac{y}{d} \right) \ldots\ldots\ldots\ldots\ldots (2)$$

in which: y = distance of stress from edge; σ_0 = unit stress at $y = 0$, the upper or lower edge of the plate; and σ_y = unit stress at distance y from edge.

The variation of stress intensity over the full width of plate can be expressed by the factor α in Eq. 2. If α is 0, the stress is constant over the entire section which is equivalent to uniform compression. If $\alpha = 2$ the plate is in pure bending. The interrelation between the stability coefficient k and the factor α can be expressed approximately by the empirical equation

$$k = \alpha^3 + 3\,\alpha^2 + 4 \ldots\ldots\ldots\ldots\ldots (3)$$

To arrive at a numerical value of the stability coefficient k, the critical stress for a given example is first computed by equating the work done by the external and internal forces. The value of the critical stress is then introduced in Eq. 1 in which the coefficient k becomes the unknown. The procedure has been well illustrated by Professor Timoshenko (41). Each example for the desired external loading must be computed for various length-to-width ratios to establish minima values of the critical stress, and thus a minimum value for the coefficient k. In order to obtain a general expression for the stress condition, an approximate formula (Eq. 3) has been created which gives minima values for k for various loading conditions.

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[4] Numerals in parentheses, thus (41*a*), refer to corresponding items in the Bibliography in the Appendix.

For uniform compression $\alpha = 0$ and $k = 4$. By introducing $k = 4$ in the fundamental formula, Eq. 1, and assuming ζ to be unity, the well-known Bryan (1) formula is derived. It is

$$\sigma_{cr} = \frac{\pi^2 E}{3\left[1 - \left(\frac{1}{m}\right)^2\right]} \left(\frac{t}{d}\right)^2 \quad \text{.....................} (4)$$

For pure bending $\alpha = 2$ and k increases to 24. It should be noted that the length-to-width ratio of the plate has been omitted in the expression for k in Eq. 3. It was tacitly assumed that the proportions of the plate are such as to make k a minimum. This is true for plate proportions which cause buckling into unrestrained half waves. For uniform compression a minimum value of k occurs in the case of a square plate. To increase k, and with it the critical buckling stress, the length of the plate must be shorter than its width. This is of little practical consideration. Plates forming parts of columns should not be supported transversely at intervals which make the distance between diaphragms less than the width of the plate. A column design with diaphragms spaced less than the width of the plates would be uneconomical. Therefore, the minimum value of the coefficient k should govern the proportioning of the plate. In other words, the influence of the length-to-width ratio of a plate on direct stresses is relatively inessential.

Plates Subject to Shear.—For plates subject to shear, the length-to-width ratio becomes of prime importance. For an infinitely long plate the coefficient k is 5.35; it increases to 9.34 for a square plate. By further decreasing the length-to-width ratio the value of k will be increased rapidly.

Table 1 gives the values of coefficient k for various length-to-width ratios, $\beta = \frac{l}{d}$, in which l is the length and d the depth of plate. Simply supported edges have been assumed for ratios of β greater than 0.50. For values of β less than 0.50, a partly clamped edge condition was assumed.

TABLE 1.—PLATES SUBJECT TO SHEAR; VALUES OF COEFFICIENT k FOR VARIOUS LENGTH-TO-WIDTH RATIOS, β

β	k	β	k	β	k	β	k	β	k	β	k
0.20	171.0	0.28	81.6	0.36	47.6	0.44	31.8	0.60	18.8	1.00	9.34
0.22	139.0	0.30	70.2	0.38	42.6	0.46	20.4	0.70	14.9	1.20	8.12
0.24	114.0	0.32	61.1	0.40	38.4	0.48	27.2	0.80	12.3	1.40	7.38
0.26	95.9	0.34	53.7	0.42	34.9	0.50	25.4	0.90	10.6	1.60	6.90

Plates Subject to Combined Direct Stresses and Shear.—The interaction of direct stresses and shear can be analyzed by separating the two strains and by expressing the effect of the shear on the critical compression.

If the shear to which the plate is subjected is as high as the critical shearing stress, it is evident that the plate cannot sustain simultaneous direct stresses, in addition. As the ratio of the actual shearing stress to its critical value decreases, the plate will retain an increasing resistance for direct stresses. This is expressed numerically in Table 2, in which the shear ratio is the ratio of the actual shear to its critical value and k_d is the percentage of k available for direct stress.

TABLE 2.—REDUCTION OF k FOR BENDING STRESSES DUE TO SIMULTANEOUS ACTION OF SHEAR

Description	SHEAR RATIOS, $\frac{\tau}{\tau_{cr}}$:									
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
Value of k_d........	0.99	0.97	0.94	0.90	0.84	0.76	0.67	0.55	0.39	0

THE MODULUS FACTOR

The modulus factor ζ (Eq. 1) is unity or less. For critical stresses below the proportional limit of the material, ζ is unity; for critical stresses above the proportional limit, ζ will be less than unity. Its magnitude depends on the critical stress and on the elastic deformation of the material. If the member in compression remains isotropic (that is, the change in elastic properties of the material is the same in all directions), the modulus factor becomes the ratio of the modulus of elasticity at the critical stress to its value below the proportional limit, denoted as $\epsilon = \frac{E_x}{E}$. For the assumption that the elastic properties change, in the direction parallel to the stress, only orthotropically, and remain the same in all other directions, the factor ζ is equal to $\sqrt{\epsilon}$.

For columns as a whole an isotropic change may be assumed to exist also above the proportional limit. A number of tests have shown this assumption to be correct; the factor ζ becomes equal to ϵ. This is not the case, however, for plates subject to compression (41*b*). For plates, it is proposed to take the average of the two extreme possibilities of deformations above the proportional limit—that is,

$$\zeta = 0.5\,(\epsilon + \sqrt{\epsilon})\;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(5)$$

The immediate problem is to establish a relation between the critical stress and E_x, or, what amounts to the same, to find the variable modulus of elasticity above the proportional limit of the material in terms of the critical stress.

A study of column strength for various slenderness ratios will lead to the desired relation. Column strength has been established by a great number of tests. These tests indicate an effective buckling length of approximately 75% of the full length of the column. Accepting this buckling length, Euler's formula may be written

$$\sigma_{cr} = \frac{\pi^2 E}{\left(0.75\,\frac{l}{r}\right)^2}\;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(6)$$

With this assumption the tests show that the proportional limit is reached at a slenderness ratio of approximately 160 for structural steels and approximately 93 for the aluminum alloy tested. For slenderness ratios below these values, the critical stress is less than given by Euler's formula with a constant modulus of elasticity.

Tests have shown, furthermore, that below a certain slenderness ratio the column strength remains practically unchanged and may be taken as equal to the yield point of the material. This slenderness ratio is about 60 for steel and 32 for the aluminum alloy. The column-strength curves connecting these limiting slenderness ratios must be tangent to the Euler curve at their upper limit and pass through the yield point at their lower limit. Tests indicate foı the connecting curve a parabola for carbon steel and straight lines for silicon steel and the aluminum alloy.[3] The empirical equations are: For carbon steel

$$\sigma_{cr} = 19{,}390 + 0.7335 \left[(162)^2 - \left(\frac{l}{r} \right)^2 \right] \qquad (7a)$$

for silicon steel

$$\sigma_{cr} = 60{,}050 - 251 \frac{l}{r} \qquad (7b)$$

and for aluminum alloy

$$\sigma_{cr} = 62{,}500 - 447 \frac{l}{r} \qquad (7c)$$

Introducing the critical stress thus computed into the modified Euler formula (Eq. 6), the value of ϵ, or the variable E, can be computed for various critical stresses. The values of ϵ and $0.5 (\epsilon + \sqrt{\epsilon})$ for various critical stresses are given in Table 3. With this information on hand the two coefficients k and ζ can be computed, and it becomes possible to apply the general expression of elastic stability, Eq. 1.

TABLE 3.—VALUES OF $\epsilon \left(= \frac{E_x}{E} \right)$ AND THE MODULUS FACTOR $\zeta\ [= 0.5 (\epsilon + \sqrt{\epsilon})]$, FOR CRITICAL STRESSES

Critical stress, σ_{cr}, in pounds per square inch	CARBON STEEL		SILICON STEEL		ALUMINUM ALLOY	
	ϵ	ζ	ϵ	ζ	ϵ	ζ
19,000	1.000	1.000	1.000	1.000	1.000	1.000
20,000	0.999	0.999	1.000	1.000	1.000	1.000
21,000	0.993	0.994	0.999	0.999	1.000	1.000
22,000	0.981	0.986	0.994	0.995	0.999	0.999
24,000	0.941	0.956	0.973	0.980	0.985	0.989
26,000	0.881	0.909	0.940	0.955	0.959	0.969
28,000	0.798	0.846	0.897	0.922	0.923	0.942
30,000	0.694	0.764	0.845	0.882	0.878	0.907
32,000	0.569	0.662	0.785	0.836	0.824	0.866
34,000	0.423	0.537	0.720	0.784	0.765	0.820
36,000	....	....	0.650	0.728	0.700	0.768
38,000	....	....	0.576	0.667	0.632	0.713
40,000	....	....	0.502	0.605	0.561	0.655
42,000	....	....	0.427	0.540	0.489	0.594

CHAPTER 2.—THICKNESS OF PLATES AND ANGLES OF COMPRESSION MEMBERS AND COMPRESSION FLANGES OF PLATE GIRDERS

The allowable ratio of plate width to plate thickness depends on the unit stress and the factor of safety desired against local buckling. It is deemed sufficient to design against local buckling with a factor of safety of 2; the critical stress, therefore, is twice the allowable stress.

The width-to-thickness ratios $\frac{d}{t}$ can be determined by the use of the fundamental equation of elastic stability, Eq. 1. Introducing $\sigma_{cr} = 2\,\sigma$ and the numerical value for $\frac{\pi^2 E}{12\left[1 - \left(\frac{1}{m}\right)^2\right]}$ into Eq. 1, the ratio $\frac{d}{t}$ becomes—

For steel:

$$\frac{d}{t} = 3{,}620\sqrt{\frac{k\,\zeta}{\sigma}} \qquad \text{(8a)}$$

and for the aluminum alloy:

$$\frac{d}{t} = 2{,}180\sqrt{\frac{k\,\zeta}{\sigma}} \qquad \text{(8b)}$$

In Eqs. (8) the stability coefficient k depends on the dimensions and end conditions of the plate. For simply supported plates the minimum value for k is 4 and the width-to-thickness ratios become—

For steel:

$$\frac{d}{t} = 7{,}240\sqrt{\frac{\zeta}{\sigma}} \qquad \text{(9a)}$$

and for the aluminum alloy:

$$\frac{d}{t} = 4{,}360\sqrt{\frac{\zeta}{\sigma}} \qquad \text{(9b)}$$

Introducing ζ for various unit stresses σ, the allowable width-to-thickness ratios are given in Table 4(*a*). The basic allowable plate slenderness is 51.7, 39.8, and 30.3 for carbon steel, silicon steel, and aluminum alloy, respectively, as given in Table 4; or, in round figures, 50, 40, and 30, respectively.

To determine the highest permissible plate slenderness, it has been customary to place the critical stress of the plates forming parts of columns equal to the yield point of the material and to use Bryan's formula (Eq. 4) with a correction factor of 0.75:

$$\sigma_p = 0.75\,\frac{\pi^2 E}{3\left[1 - \left(\frac{1}{m}\right)^2\right]}\left(\frac{t}{d}\right)^2 \qquad \text{(10)}$$

For carbon steel with a yield point of 36,000 lb per sq in., Eq. 10 gives a ratio of $\frac{d}{t} = 46.7$, and, for silicon steel with a yield point of 45,000, a ratio of 41.8. This method of computing the permissible plate slenderness is somewhat arbi-

TABLE 4.—WIDTH-TO-THICKNESS RATIOS, $\frac{d}{t}$

Material	RATIOS FOR THE FOLLOWING VALUES OF THE ALLOWABLE STRESS σ, IN KIPS PER SQUARE INCH										
	(a) COLUMN PLATES										
	10	11	12	13	14	15	16	17	18	19	20
Carbon steel	72.4	68.5	64.6	60.5	56.3	51.7	....	....	....	....	....
Silicon steel	72.4	68.9	65.4	62.1	58.8	55.5	52.3	49.2	46.0	42.9	39.8
Aluminum alloy	43.6	41.6	39.6	37.6	35.8	33.9	32.1	30.3	....	....	....
Material	(b) OUTSTANDING PARTS OF COLUMNS										
	10	11	12	13	14	15	16	17	18	19	20
Carbon steel	24.0	22.7	21.5	20.1	18.6	17.1	....	....	....	....	....
Silicon steel	24.0	22.8	21.6	20.6	19.5	18.4	17.3	16.3	15.3	14.2	13.2
Aluminum alloy	14.0	13.3	12.7	12.1	11.5	10.9	10.3	9.7	....	....	....
Material	(c) BEAM AND GIRDER FLANGES										
	14	15	16	17	18	19	20	21	22	23	24
Carbon Steel:											
Rolled or riveted	46.6	42.8	39.0	35.2	31.5	....	....	....	....	....	....
Welded	42.0	38.6	35.2	31.7	28.3	....	....	....	....	....	....
Silicon Steel:											
Rolled or riveted	48.7	46.0	43.4	40.7	38.2	35.5	33.0	30.7	28.3	26.0	23.6
Welded	43.9	41.4	39.0	36.7	34.4	32.0	29.8	27.7	25.5	23.4	21.2
Aluminum Alloy:											
Rolled or riveted	28.7	27.2	25.8	24.3	22.9	21.4	20.0	18.6	....	....	....

trary; but, as seen, the results do not considerably deviate from the more refined analysis.

Eq. 1 can also be used to determine the buckling strength of outstanding parts of columns. These parts are held at one edge only. For this condition the value of the stability coefficient k is considerably smaller compared to its value for plates supported along two edges. The value of k varies between 0.43 and 1.28 for steel and between 0.41 and 1.25 for the aluminum alloy. The lower values are for a simply supported edge (that is, the outstanding part can rotate freely); the higher values are for a fixed edge. For an elastically built-in edge the value of k is between the foregoing limits. To allow for initial curvature the minimum values of k will be taken. The ratios $\frac{d}{t}$ then become—

For steel:

$$\frac{d}{t} = 2{,}400\sqrt{\frac{\zeta}{\sigma}} \qquad \text{(11}a\text{)}$$

and for the aluminum alloy:

$$\frac{d}{t} = 1{,}400\sqrt{\frac{\zeta}{\sigma}} \qquad \text{(11}b\text{)}$$

For various values of σ the allowable width-to-thickness ratios are given in

Table 4(b). Fillets increase the local buckling stability of the outstanding legs of angles. It was found that the width of the outstanding leg can be reduced by twice the thickness and this width used for determining the allowable $\frac{d}{t}$-ratio. Eq. 1 may also be applied to study the local buckling behavior of the compression flanges of rolled beams and plate girders.

The buckling behavior of compression flanges between lateral supports is vastly different from that of columns subject to axial stress. In columns, the entire section is in compression, and the sectional properties fully define the buckling criterion. In beams, half the section is in tension. This has a great stabilizing effect on the buckling of the compression flanges. The allowable stress in beams will be governed, in all practical cases, by the buckling of the compression flanges. The buckling resistance is increased materially by that part of the section that is in tension; the allowable stress, therefore, can be taken higher than is permitted for columns to result in equal factors of safety. This is recognized in prevailing specifications.

It is evident that the compression flanges must be designed also to assure local buckling stability for the increased allowable stresses. A distinction must be made between rolled, riveted, and welded beams.

Fillets of rolled beams increase considerably the local buckling strength of the outstanding flanges. It was found that the width-to-thickness ratio of the outstanding flange can be increased by approximately 25%. Considering the full width of the compression flange, this ratio must be doubled. Solving Eq. 1 for these conditions, the allowable width-to-thickness ratio of compression flanges of rolled beams will be—

For steel:

$$\frac{d}{t} = 6{,}000\sqrt{\frac{\zeta}{\sigma}} \quad \text{(12a)}$$

and for the aluminum alloy:

$$\frac{d}{t} = 3{,}500\sqrt{\frac{\zeta}{\sigma}} \quad \text{(12b)}$$

in which d = depth of flange and t = thickness of flange.

For riveted plate girders the same expression may be applied. If the flanges consist of angles, only the fillets will stiffen the outstanding leg. If the flanges consist of riveted angles and cover plates, the actual compressive stress is considerably less than the computed stress, which is based on the net section. In Eqs. 8, 9, 11, and 12, σ is the unit stress on the net section and ζ the corresponding modulus factor. Adhering to the allowable unit stresses of prevailing specifications, the maintenance of a factor of safety of two is not possible in every case because the doubled values of the allowable stresses exceed the yield stress, beyond which ζ is theoretically zero. Factors of safety of two are maintained up to a stress of 15,000 lb per sq in. for carbon steel and 18,000 lb per sq in. for silicon steel. From these values the factors of safety gradually decrease to 1.92 and 1.85 for the maximum allowed stresses of 18,000 and 24,000 lb per sq in. for carbon and silicon steel, respectively. For the aluminum alloy, a factor of safety of two is maintained for all stresses.

In welded girders the allowable unit stress is fully realized. Although the flanges may be sheared from rolled beams, the fillets may not (in all cases) have the full effect of the rolled section. Therefore, a reduction in the allowable width-to-thickness ratio should be made. It is proposed to use

$$\frac{d}{t} = 5{,}400\sqrt{\frac{\zeta}{\sigma}} \qquad \text{(13)}$$

Eqs. 12 and 13 have been used in preparation of Table 4(*c*).

CHAPTER 3.—DESIGN OF WEBS FOR PLATE GIRDERS

The buckling behavior of the webs of plate girders is more complicated than that of plates forming parts of columns. In the latter case uniform compression is the rule; in the former it is the exception. Plates in columns form a main portion of the cross-sectional area, and their buckling will cause failure of the member. It is mainly for this reason that the critical buckling stress was placed rather close to the yield point. In plate girders the webs contribute little area to the compression flange. The buckling of the web causes its proportionate share of compression to be transferred to the flange and, since this total stress is small, it will cause a correspondingly small overstress in the compression flange. Furthermore, the buckled web adjusts itself to sustain the shear by tension and transmit it to the vertical stiffener. Buckling of the web of plate girders, therefore, will not cause failure of the structure, and for these reasons a lower factor of safety against buckling of the web is justified.

Factors of safety of 1.5 for compression in flexure or shear and of 1.4 for the combined action of compressive flexure and shear against buckling of webs of plate girders were adopted. These factors of safety are considered adequate and equivalent to a factor of safety of 2 against buckling of plates forming parts of columns.

The buckling strength of the web depends on the magnitude of the axial stress at the toes of the flange angles and the amount of the average shear acting in combination with compression. The problem becomes rather complex and the allowable width-to-thickness ratio of the web (which, herein, is conveniently called "web slenderness") cannot be divorced from the magnitude of axial stress and shear. The effect of these will be discussed in more detail in Chapter 4. A simple example of pure bending will be stated in the present chapter, however.

The stress condition of pure bending prevails at the center of a simply supported plate girder symmetrically loaded where the shear is practically zero; the buckling coefficient k thus has the full value of 24. The greatest allowable bending stress in tension is 18,000 lb per sq in. for carbon steel, 24,000 lb per sq in. for silicon steel, and 21,000 lb per sq in. for the aluminum alloy. The corresponding tension at the toe of the flange angles for average girders is 15,000, 20,000, and 17,000 lb per sq in., respectively. These stresses should also be applied for compression at the toe of the flange angles to be in accord with prevailing specifications which require the same net section for the tension as well as the compression side. The allowable critical buckling stresses, with a factor of safety of 1.5, therefore, will become 22,500 lb per sq in. for carbon steel, 30,000 lb per sq in. for silicon steel, and 25,500 lb per sq in. for the aluminum alloy. The corresponding values of ζ from Table 3 are 0.98, 0.88, and 0.98, respectively. For these values Eq. 1 results in web slenderness values of 165, 136, and 94. It will be noted that the resulting values are in general accord with standard specifications.

CHAPTER 4.—PLATES IN COLUMNS AND WEBS OF GIRDERS REINFORCED BY STIFFENERS

The principal function of stiffeners consists in increasing the buckling resistance of the plates to which they are attached. The stiffeners divide the plates or webs into panels, and it is evident that their economical usefulness demands such proportioning that the critical stress of the entire structure is equal to the critical stress of the most stressed panel. The dimensioning of the stiffeners must be such that they form nodal lines at their locations when the critical stress is reached.

To increase the size of the stiffeners above the minimum requirements will not increase the critical resistance of the structure because it cannot be stronger than its weakest panel. If rules could be established for the size of the stiffeners to assure equivalent buckling stability, each individual panel could be treated as a unit. For such units equations and tables have already been established in the previous discussion of the buckling stability of plates (Chapter 2).

Plates forming parts of columns, and reinforced by equidistant stiffeners, permit an extreme plate slenderness of $\frac{d}{t}(N+1)$, in which $\frac{d}{t}$ is the basic ratio 50, 40, and 30 for carbon steel, silicon steel, and the aluminum alloy, respectively, and N is the number of stiffeners. The basic web slenderness of plate girders can be increased in a similar manner by the placing of longitudinal stiffeners at proper locations.

A mathematical analysis of the integrated buckling stability of plates or webs reinforced by stiffeners is rather involved. A study of the available literature will show the heavy mathematical "artillery" necessary to arrive at an approximate solution for simple assumptions.

It may be stated that the size of the stiffeners depends generally on the thickness of the plate they must stiffen; increasing the thickness of plate demands increasing the size of stiffeners. Axial load controls the dimensioning of stiffeners in the direction of its action, whereas stiffeners normal to the axial load are governed by shear.

GENERAL THEORY OF LONGITUDINAL STIFFENERS IN PLATES OF COLUMNS AND HORIZONTAL STIFFENERS IN WEBS OF PLATE GIRDERS

The buckling strength of the individual panels into which the stiffeners divide the plate or web and of the integrated structure is expressed by Eq. 1. Knowing the dimensions of each panel and the stress conditions to which each is subjected, its critical stress can be computed. This stress should also be the critical stress of the reinforced plate or web. Since the dimensions and proportions of the latter are known, the equivalent stability coefficient k_e of the reinforced structure can be computed numerically from Eq. 1 as applied to the reinforced structure. The procedure, therefore, is to establish an expression for the equivalent coefficient k_e for the reinforced plate or web which will result in the same critical stresses as computed for the individual panels.

The rigidity of a plate R_p is

$$R_p = \frac{E\,t^3\,d}{12\left[1 - \left(\frac{1}{m}\right)^2\right]} \ldots\ldots\ldots\ldots\ldots\ldots (14a)$$

and the rigidity of the stiffener is

$$R_s = E\,I_s \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (14b)$$

in which I_s = moment of inertia of the stiffener. For a stiffener on one side of the plate only, the value of I_s is to be taken along the inner face of the plate; for stiffeners on both sides, I_s is to be taken about the center of the plate. The ratio δ_s of the area of stiffener to the area of plate is

$$\delta_s = \frac{A_s}{t\,d} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (15a)$$

and the ratio γ_s of rigidity of stiffener to rigidity of plate is

$$\gamma_s = \frac{12\left[1 - \left(\frac{1}{m}\right)^2\right]}{t^3\,d}\,I_s \ldots\ldots\ldots\ldots\ldots\ldots (15b)$$

The buckling strength of each panel is expressed by Eq. 1 in which d is now the width of the panel. The coefficient k can be computed for any known stress condition, as shown in Chapter 1. The buckling strength of the reinforced plate or web is equally expressed by Eq. 1 in which d now becomes the total width of the member. Since both critical stresses must be the same, a numerical value for the equivalent coefficient k_e can be computed as follows:

$$k_e = k'\left(\frac{d'}{d_s}\right)^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

in which k' = coefficient for the individual panel, d' = width of reinforced structure, and d_s = width of individual panel.

Coefficient k' is only a function of the panel dimensions and stress conditions. The equivalent coefficient is not only dependent on the dimensions of the member and the stress conditions, but also becomes a function of the ratios δ_s and γ_s. A general approximate value is:

$$k_e = \frac{k_0}{4}\left\{\frac{(1+\beta^2)^2 + 2\,\Sigma\gamma_s \sin^2(\pi\,c_s)}{\beta^2\left[1 + 2\,\Sigma\delta_s \sin^2(\pi\,c_s)\right]}\right\} \ldots\ldots\ldots\ldots (17)$$

in which k_0 = stability coefficient for the unreinforced plate, β = ratio of $\frac{l}{d}$, l = length of plate between transverse stiffeners, and c_s = ratio of distance of stiffener from edge of plate to width of plate. With $r^2 = \frac{I_s}{A_s}$ and $\delta_s = \frac{A_s}{t\,d}$, the value for γ_s can be written

$$\gamma_s = 12\left[1 - \left(\frac{1}{m}\right)^2\right]\delta_s\left(\frac{r}{t}\right)^2 \ldots\ldots\ldots\ldots\ldots(18)$$

Substituting in the expression for k and solving for $\left(\frac{r}{t}\right)^2$,

$$\left(\frac{r}{t}\right)^2 = \frac{4\frac{k_e}{k_0}\beta^2[1 + 2\,\Sigma\delta_s \sin^2(\pi c_s)] - (1 + \beta^2)^2}{24\left[1 - \left(\frac{1}{m}\right)^2\right]\Sigma\delta_s \sin^2(\pi c_s)} \ldots\ldots\ldots(19)$$

For maxima values of $\left(\frac{r}{t}\right)^2$ the differential $\frac{\partial\left(\frac{r}{t}\right)^2}{\partial\beta}$ must be zero; β then becomes:

$$\beta = \left\{2\frac{k_e}{k_0}[1 + 2\,\Sigma\delta_s \sin^2(\pi c_s)] - 1\right\}^{0.5} \ldots\ldots\ldots\ldots(20)$$

It is seen from Eq. 19 that the value of $\frac{r}{t}$ depends not only on the ratio β and the location and the number of stiffeners, but also on the value of δ_s which is the ratio of stiffener area to area of plate. Since the ratio δ_s affects both sides of the equation, an explicit form for $\frac{r}{t}$ can be solved only by introducing numerical values for the ratio δ_s. In evaluating this ratio it should be noted that, with decreasing values, the expression for $\frac{r}{t}$ in Eq. 19 increases. To establish requirements for the ratio $\frac{r}{t}$, such stiffeners should be assumed in the computations which give the largest radius of gyration with the minimum amount of section.

In these computations the smallest possible values for the ratio δ_s were used. The smallest values are obtained by using stiffeners which give the greatest radius of gyration about the face of the plate or web with the least amount of sectional area. This limiting relation is termed here "highest stiffener efficiency." The relation between the radius of gyration and the area of stiffener is shown in Fig. 1. These curves condense the results of a large number of computations using available rolled shapes. Bulb angles were found to be most efficient for riveted work; bars should be used for small welded sections; and tees with the stem welded to the plate should be used for large welded sections.

Should stiffeners be used which are not as efficient as shown by the curves in Fig. 1, the ratio of δ_s will decrease accordingly. The requirements for the size of stiffeners are based on minima values of δ_s. For values of δ_s above the minimum, the required radius of gyration is slightly smaller. This reduction in the required radius of gyration is fairly negligible. It is advisable to use minima values for δ_s. This becomes clear in the application to practical cases.

In columns the stiffeners sustain their proportional share of compression and assist in stiffening the plate. No waste of material is caused by using a heavier stiffener of low efficiency. In plate girders the horizontal stiffeners are purely stiffeners, and the highest efficiency should be effected.

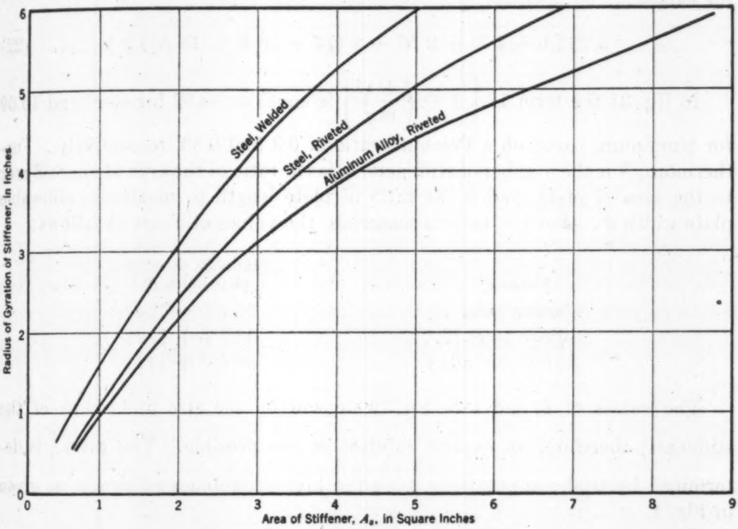


FIG. 1.—HIGHEST STIFFENER EFFICIENCY

LONGITUDINAL STIFFENERS IN PLATES OF COLUMNS

The proper spacing and proportioning of longitudinal stiffeners increase the allowable plate slenderness to multiples of the unreinforced column plate, depending on the number of stiffeners used. This warrants the use of thinner plates and allows a greater part of the material to be placed on the periphery of the column where it is most effective. In cellular structures, such as towers of suspension bridges, or large posts of other bridges, the use of longitudinal stiffeners permits larger cells. This simplifies the fabrication and erection of the structures.

TABLE 5.—TERMS FOR USE IN EQS. 19 AND 20 WHEN COMPRESSION IS UNIFORM AND STIFFENERS ARE EQUIDISTANT

Description	NUMBER OF STIFFENERS, N		
	1	2	3
Equivalent coefficient, k' (for each case $k_0 = 4$)	16	36	64
$\Sigma \delta_s \sin^2 \pi c_s$ (see Eq. 20)	δ	1.5δ	2.0δ

Size and Spacing of Longitudinal Stiffeners.—For uniform compression and equal and equidistant stiffeners, the terms in Eqs. 19 and 20 are as shown in

Table 5. With these established values, Eq. 19 can be written,

$$\frac{r}{t} = \left\{ \frac{\beta^2 [2 + 8\,N + 4\,N^2 + \delta_s (28 - 32\,N + 36\,N^2)] - \beta^4 - 1}{12\,[1 - (1/m)^2]\,(N + 1)\,\delta_s} \right\}^{0.5} \qquad (21)$$

for values of

$$\beta \gtreqless [1 + 4\,N + 2\,N^2 + \delta\,(14 - 16\,N + 18\,N^2)]^{0.5} \ldots\ldots \qquad (22)$$

In Eq. 21 the term $12 \left[1 - \left(\frac{1}{m} \right)^2 \right]$ is equal to 10.92 for steel and 10.69 for aluminum, based on a Poisson's ratio of 0.3 and 0.33, respectively. Furthermore, N is the number of stiffeners, δ_s is the ratio of the area of one stiffener to the area of plate, and β the ratio of plate length to maximum allowable plate width d'. For the various materials, the values of d' are as follows:

Material	Maximum allowable plate width, d'
Carbon steel	$50\,(N + 1)\,t$
Silicon steel	$40\,(N + 1)\,t$
Aluminum alloy	$30\,(N + 1)\,t$

The values of δ_s and r in Eq. 19 depend on the size and shape of the stiffeners; therefore, an explicit solution is not possible. The ratio $\frac{r}{t}$ is determined by trial computations using the highest stiffener efficiency, as given in Fig. 1.

For given values of β, t, and N, a value of r is assumed and the corresponding stiffener area is taken from Fig. 1. The value of δ_s is then the area of the stiffener divided by the area of the plate, $d' \times t$. Introducing this value of δ_s into Eq. 21, a value of r will be obtained. The computation should be repeated until the computed value of r agrees with the assumed value. Table 6, which gives the required radius of gyration for various plate thicknesses and values of β, was obtained in this manner and is presented to facilitate computations.

Eq. 20, which expresses values for β for maxima values of $\frac{r}{t}$, indicates that for these limiting ratios no transverse stiffeners will be useful. If the length l is such that β will satisfy Eq. 20, the stiffened plate will buckle in an unrestrained half wave, and a transverse stiffener at these locations will not help to increase the critical buckling stress.

It should be noted that, in the derivation of Eq. 20, the value of δ_s was assumed to be independent of β. It was found that the theoretically correct values of β for maxima values of $\frac{r}{t}$ deviate but slightly from those given by Eq. 20, which justifies the use of this equation in its simplified form. Eq. 20 can be transformed to express minima values of $\frac{r}{t}$ for which no transverse stiffeners are required. The transformed equation reads—

TABLE 6.—RADIUS OF GYRATION REQUIRED PER STIFFENER IN INCHES FOR COLUMN PLATES OF VARIOUS THICKNESSES t AND RATIOS β AND REINFORCED BY ONE, TWO, OR THREE LONGITUDINAL STIFFENERS

Ratio $\beta = l/d$	THICKNESS, t, OF COLUMN PLATE, IN INCHES											
	3/8			1/2			5/8			3/4		
	Number of Stiffeners, N											
	1	2	3	1	2	3	1	2	3	1	2	3
(*a*) CARBON STEEL; RIVETED												
1.0	1.12	1.60	2.01	1.67	2.34	2.91	2.25	3.12	3.82	2.85	3.90	4.74
1.5	1.49	2.15	2.68	2.19	3.09	3.82	2.92	4.06	4.94	3.66	4.98	5.97
2.0	1.78	2.61	3.28	2.58	3.71	4.59	3.40	4.80	5.83	4.24	5.82	7.09
2.5	1.97	3.01	3.81	2.83	4.25	5.25	3.71	5.40	6.64	4.59	6.56	8.14
3.0	2.05	3.35	4.28	2.91	4.67	5.83	3.79	5.91	7.42	4.67	7.20	9.11
(*b*) SILICON STEEL; RIVETED												
1.0	1.04	1.50	1.88	1.55	2.19	2.73	2.10	2.92	3.60	2.66	3.66	4.47
1.5	1.41	2.01	2.53	2.05	2.90	3.61	2.73	3.82	4.68	3.44	4.72	5.68
2.0	1.68	2.46	3.10	2.42	3.51	4.38	3.20	4.55	5.54	4.00	5.53	6.74
2.5	1.86	2.85	3.62	2.67	4.02	5.02	3.51	5.16	6.33	4.34	6.23	7.75
3.0	1.95	3.18	4.10	2.76	4.41	5.59	3.60	5.65	7.09	4.44	6.86	8.68
(*c*) ALUMINUM ALLOY; RIVETED												
1.0	0.94	1.35	1.70	1.40	1.98	2.47	1.88	2.63	3.23	2.38	3.27	3.98
1.5	1.27	1.83	2.32	1.85	2.63	3.27	2.46	3.43	4.19	3.08	4.20	5.05
2.0	1.53	2.25	2.85	2.21	3.18	3.95	2.89	4.08	4.98	3.59	4.93	6.02
2.5	1.72	2.62	3.33	2.45	3.65	4.56	3.19	4.64	5.72	3.93	5.57	6.94
3.0	1.82	2.94	3.78	2.57	4.06	5.09	3.32	5.11	6.43	4.06	6.17	7.83
(*d*) CARBON STEEL; WELDED												
1.0	1.25	1.75	2.17	1.82	2.53	3.13	2.43	3.36	4.12	3.08	4.21	5.09
1.5	1.63	2.31	2.90	2.36	3.32	4.10	3.14	4.36	5.29	3.95	5.35	6.48
2.0	1.92	2.79	3.50	2.77	3.98	4.92	3.66	5.15	6.24	4.56	6.25	7.60
2.5	2.11	3.21	4.05	3.03	4.53	5.60	3.98	5.79	7.10	4.91	7.02	8.69
3.0	2.17	3.55	4.55	3.08	4.98	6.21	4.03	6.32	7.90	4.97	7.69	9.69
(*e*) SILICON STEEL; WELDED												
1.0	1.17	1.64	2.04	1.70	2.37	2.93	2.27	3.15	3.88	2.89	3.96	4.81
1.5	1.53	2.17	2.72	2.22	3.12	3.87	2.94	4.10	5.03	3.71	5.07	6.10
2.0	1.80	2.64	3.31	2.60	3.75	4.66	3.44	4.88	5.94	4.28	5.95	7.20
2.5	2.00	3.04	3.85	2.86	4.29	5.34	3.75	5.48	6.74	4.65	6.68	8.24
3.0	2.06	3.38	4.34	2.93	4.73	5.94	3.83	6.02	7.52	4.67	7.31	9.21

For steel:

$$\frac{r}{t} = (4 + 2\,N^2)\,\frac{d}{d'} \qquad \text{(23a)}$$

and for aluminum alloy:

$$\frac{r}{t} = (2.5 + 3\,N^2)\,\frac{d}{d'} \qquad \text{(23b)}$$

in which d is the actual and d' the basic allowable plate width.

It will be noted that the theoretical plate width is introduced in both Eqs. 21 and 23. This was done to adapt the equations to practical requirements. In the actual design of columns the full allowable plate width can seldom be applied because other limitations have to be considered to effect a balanced structure. Thus it often may be the case that the longitudinal stiffeners are spaced closer than theoretically required. This will increase the buckling stability of the panel above the required limits; but there is no reason why the stiffened plate as a whole should have the same increased buckling strength.

TRANSVERSE STIFFENERS IN PLATES OF COLUMNS

In the analysis of the behavior of plates reinforced by longitudinal stiffeners, it was assumed that the transverse stiffeners will prevent buckling at their locations. A method had to be found to determine the size of transverse stiffeners to act as nodal lines. A mathematical approach to this problem is rather involved, and it is doubtful that an exact analytical solution is possible. The procedure adopted to approach a solution of this problem follows from the knowledge of the buckling behavior of plates.

An unreinforced plate subject to uniform compressive stresses will buckle into waves; the number of these waves depends on the ratio of length to width of plate. It is evident that if a transverse stiffener is placed at the nodal line of the plate the latter will not increase its buckling strength. However, should transverse stiffeners be placed in such a manner that they will shorten the length of each half wave, the buckling stability will be increased. Therefore, the problem was approached as follows:

A panel of a column plate reinforced by longitudinal stiffeners was virtually transformed into an orthogonally isotropic plate of the same width but of a reduced length. The buckling resistance of a panel of length l and depth d is proportional to its rigidity in longitudinal and transverse directions and, inversely, proportional to the fourth power of the dimensions of l and d. This can be deduced from the deflections of strips of the plate acting at right angles to each other; at their intersection the deflections must be identical. To transform the panel into a virtual plate of equal rigidities in both directions, the length l, should be reduced by the ratio ρ. It also can be deduced from this that if the actual ratio β of the panel is such that it is equal to ρ the least critical stress will be obtained. In other words, transverse stiffeners placed in accordance with this expression will not increase the buckling stability of the plate.

A number of examples have been computed and it was found that a general approximation for the ratio $\frac{1}{\rho}$ may be expressed as a function of β and N, namely,

$$\frac{1}{\rho} = 0.50 \sqrt{\frac{\beta}{N}} \frac{1}{\beta} \qquad (24)$$

This relation makes it possible to establish the virtual ratio $\beta' = \frac{\beta}{\rho}$. The panel can thus be transformed into an orthogonally isotropic plate with a length-to-width ratio of β'.

It is reasoned that, if it is reinforced by transverse stiffeners, this plate should develop the same critical stress as the actual plate reinforced by longitudinal stiffeners. For the actual panel the general expression for the critical stress, Eq. 1, is

$$\sigma_{cr} = k_1 \frac{\pi^2 E \zeta}{12\left[1 - \left(\frac{1}{m}\right)^2\right]} \left(\frac{t}{d}\right)^2 \ldots\ldots\ldots\ldots\ldots (25a)$$

and, for the virtual plate between transverse stiffeners,

$$\sigma_{cr} = k_2 \frac{\pi^2 E \zeta}{12\left[1 - \left(\frac{1}{m}\right)^2\right]} \left(\frac{t_v}{d}\right)^2 \ldots\ldots\ldots\ldots\ldots (25b)$$

in which t_v is the virtual thickness. The value k_1 is 16, 36, and 64 for $N = 1$, 2, and 3, respectively. The value k_2 can be established for various values of β' or β.

By equating the two values of the critical stress, an expression for t_v can be established. From $k_1 = 4\,(N + 1)^2$, and $k_2 = \left(0.50\sqrt{\frac{\beta}{N}} + 2.0\sqrt{\frac{N}{\beta}}\right)^2$,

$$t_v = \frac{2\,(N + 1)}{0.50\sqrt{\frac{\beta}{N}} + 2.0\sqrt{\frac{N}{\beta}}}\, t \ldots\ldots\ldots\ldots\ldots (26)$$

A close approximation for t_v is given by

$$t_v^3 = 4\,\beta\, N\, t^3 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (27)$$

The ratio of the rigidity of a transverse stiffener to the virtual plate is

$$\gamma_v = \frac{12\left[1 - \left(\frac{1}{m}\right)^2\right]}{t_v^3\, d}\, I_t \ldots\ldots\ldots\ldots\ldots\ldots\ldots (28)$$

or, substituting Eq. 27 in Eq. 28, the ratio becomes:

$$\gamma_v = \frac{12\left[1 - \left(\frac{1}{m}\right)^2\right]}{4\,\beta\, d N\, t^3}\, I_t \ldots\ldots\ldots\ldots\ldots\ldots\ldots (29)$$

An approximate critical value of this ratio can theoretically be established for any number of transverse stiffeners. In practical design the value of β' is between 0.3 and 0.7. For this range an approximate expression for γ_v is

$$\gamma_v = \frac{0.018\left\{12\left[1 - \left(\frac{1}{m}\right)^2\right]\right\}}{(\beta')^3} \sqrt{N'} \ldots\ldots\ldots\ldots\ldots (30)$$

in which N' = number of transverse stiffeners and $\beta' = \frac{\beta}{\rho}$.

Each stiffener should be so proportioned that the value of γ_v will not be less than given by Eq. 30. A combination of Eqs. 29 and 30 will give an approximate value for the moment of inertia I_t required for one transverse stiffener:

$$I_t = 0.58\sqrt{\frac{N^5}{\beta}}\, t^3 d \sqrt{N'} = T\, t^3 d \sqrt{N'} \qquad (31)$$

for values of $\beta \lesseqgtr 3.2\,N$. The factor T for various values of β and N' is given in Table 7.

TABLE 7.—VALUES OF COEFFICIENT T, IN EQ. 31

Number of stiffeners, N	COEFFICIENT T FOR THE FOLLOWING VALUES OF $\beta = l/d$:												
	0.8	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	3.5	4.0	4.5	5.0
1	0.648	0.580	0.529	0.490	0.459	0.432	0.410	0.367	0.335	....	....	....	....
2	3.67	3.34	3.00	2.77	2.60	2.44	2.32	2.08	1.90	1.75	1.64	1.54	1.47
3	10.10	9.04	8.26	7.64	7.16	6.73	6.39	5.72	5.22	4.83	4.52	4.25	4.04

Eq. 31 and Table 7 apply to steel as well as to aluminum. The table indicates smaller transverse stiffeners with an increasing ratio β. This is in accord with the theory. Denoting the unit rigidity of a plate in a longitudinal (R_a) and a transverse (R_b) direction, respectively, by:

$$R_a = \frac{E\,t^3}{12\left[1 - \left(\frac{1}{m}\right)^2\right]} + \frac{E\,I_s}{b_w\,t} \qquad (32a)$$

and

$$R_b = \frac{E\,t^3}{12\left[1 - \left(\frac{1}{m}\right)^2\right]} \qquad (32b)$$

no transverse stiffeners will be required if $\beta = \sqrt[4]{\frac{R_a}{R_b}}$. With a decreasing ratio, β, the required size of the transverse stiffeners increases. This is also in accord with the theory because it forces the plate into shorter half waves.

VERTICAL STIFFENERS OF WEBS OF PLATE GIRDERS

Vertical stiffeners or intermediate stiffeners of plate girders prevent the web from buckling below the desired critical stress caused by shear. Shear is fundamentally a tension stress and can be compared to pure tension with the understanding that the yield strength for shear is below the yield strength in tension. For steels this reduction is usually taken to be 36%, resulting in a yield strength of 23,000 lb per sq in. for carbon steel and 28,800 lb per sq in. for silicon steel; the corresponding value for the aluminum alloy is 30,000 lb per sq in. The proportional limit is about 80% of the yield strength; the same reduction may be taken for shear. The values for the proportional limits in shear are: For carbon steel 18,400, for silicon steel 23,000, and for the aluminum alloy 24,000, lb per sq in.

The moduli of elasticity in shear below the values representing the proportional limit are constant. Above the proportional limit the moduli will decrease, theoretically, approaching zero at the yield point. The transition curve between the proportional limit and the yield point was assumed to be an ellipse as the best approximation derived from the shear stress-strain curves of the materials. With these assumptions the value of ζ can be computed for various critical stresses. The results are compiled in Table 8. The allowable unit shear for carbon steel is 11,000, for silicon steel 14,000, and for the aluminum alloy, 12,500, lb per sq in. With a factor of safety of 1.5, the critical shearing stresses become 16,500, 21,000, and 18,750, lb per sq in., respectively.

TABLE 8.—VALUES OF MODULUS FACTOR ζ FOR VARIOUS CRITICAL STRESSES

Meta	MODULUS FACTOR ζ FOR THE FOLLOWING VALUES OF CRITICAL SHEARING STRESS, τ_{cr}, IN KIPS PER SQUARE INCH:											
	18	19	20	21	22	23	24	25	26	27	28	29
Carbon steel	1.000	0.994	0.953	0.867	0.706	....	....	....	....	....	....	....
Silicon steel	1.000	1.000	1.000	1.000	1.000	1.000	0.989	0.954	0.890	0.788	0.609	....
Aluminum alloy	1.000	1.000	1.000	1.000	1.000	1.000	1.000	0.990	0.957	0.898	0.804	0.648

Spacing of Vertical Stiffeners.—It will be noted that the proportional limit for each material is greater than the foregoing critical buckling stress. Therefore, the value of ζ in Eq. 1 can be taken as unity; in other words, the modulus of elasticity is constant for shear, 29,000,000 lb per sq in. for steel, and 10,300,000 lb per sq in. for aluminum.

In Eq. 1, let: d = width of web between the toes of the flange angles, l_s = the spacing of intermediate stiffeners, $\beta = \dfrac{l_s}{d}$, τ = unit shear in web, and τ_{cr} = critical shearing stress = $1.5\,\tau$. Then, solving for l_s,

$$l_s = \frac{\beta\, t \left\{ k \dfrac{\pi^2 E}{12\,[1 - (1/m)^2]} \right\}^{0.5}}{(1.5\,\tau)^{0.5}} \quad\ldots\ldots\ldots\ldots\ldots\ldots(33)$$

The ratio of the modulus of elasticity for shear to the tensile modulus is included in the value of k used in Eq. 33 (see heading "General Expression for Elastic Stability"). Table 1 gives values of k for various ratios β. Introduced in Eq. 33, the expression for l_s can be written

$$l_s = \text{coefficient} \times \frac{t}{\sqrt{\tau}} \quad\ldots\ldots\ldots\ldots\ldots\ldots(34)$$

The values of the coefficient for various ratios β are:

β	For steels	For aluminum
0.2	10,910	6,570
0.4	10,370	6,240
0.6	10,870	6,540
0.8	11,700	7,050
1.0	12,750	7,680

It will be noted that the coefficient changes little for various values of β; and, therefore, it is justifiable to eliminate β for practical requirements. With this in view, it is proposed to use, for the intermediate stiffener spacing, the following simple expressions—

For steel:

$$l_s = 10{,}500 \frac{t}{\sqrt{\tau}} \qquad \text{(35}a\text{)}$$

and for the aluminum alloy:

$$l_s = 6{,}500 \frac{t}{\sqrt{\tau}} \qquad \text{(35}b\text{)}$$

Not all girder webs require intermediate vertical stiffeners, which can be demonstrated by applying the general stability expression, Eq. 1. For long, simply supported plates the value of k is 5.35. Taking the critical shearing stress as one and one half times the actual shear, the modulus factor ζ is unity, and the clear depth, d (for which no intermediate stiffeners are required) becomes

For steel:

$$d = \frac{9{,}600\, t}{\sqrt{\tau}} \qquad \text{(36}a\text{)}$$

and for aluminum:

$$d = \frac{5{,}800\, t}{\sqrt{\tau}} \qquad \text{(36}b\text{)}$$

In these expressions the factors 9,600 and 5,800 have been "rounded out" from 9,669 and 5,823, respectively. Table 9 gives a few values of the maximum allowable clear depth of webs for which no intermediate stiffeners are required.

TABLE 9.—MAXIMUM ALLOWABLE CLEAR DEPTH, d, IN INCHES, OF WEBS REQUIRING NO INTERMEDIATE STIFFENERS

Material	Thickness, t, in inches	DEPTHS, IN INCHES, FOR THE FOLLOWING VALUES OF WEB STRESS, τ, IN KIPS PER SQUARE INCH:							
		1	2	3	4	5	6	8	10
Steel	3/8	113.8	80.5	65.7	56.9	50.9	46.5	40.3	36.0
Aluminum	3/8	68.8	48.6	39.7	34.4	30.8	28.1	24.3	20.7
Steel	1/2	151.8	107.3	87.6	75.9	67.9	62.0	53.7	48.0
Aluminum	1/2	91.7	64.8	52.9	45.9	41.0	37.4	32.4	27.7
Steel	5/8	189.7	134.2	109.5	94.9	84.8	77.5	67.1	60.3
Aluminum	5/8	114.6	81.1	66.2	57.3	51.3	46.8	40.5	34.6

Size of Vertical Stiffeners.—To determine the required size of vertical stiffeners it should be kept in mind that they may be called upon to transmit load concentrations, or shear, or both, and act as stiffeners as well. In the first case, depending on the load concentration, a symmetrical stiffener may be of advantage, whereas in the latter case a stiffener on one side of the web only will show greater efficiency. It is prudent, therefore, to design the vertical stiffeners as columns and as stiffeners. The column should be assumed to consist of the stiffener and a strip of web equal to twenty-five times its thickness; the column length may be taken at three quarters of the actual length. To assure stability under all conditions, the load taken for the assumed column should consist of the load concentration and part of the shear, depending on the spacing of the stiffener. The following load is proposed:

$$P = V\beta + P_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (37)$$

but not more than $(V + P_1)$. In Eq. 37: V = average of values of shear at left and right side of stiffener, in lb; P_1 = local load concentration, in lb; and β = ratio of stiffener spacing to total height of web (not to exceed 1.0).

A mathematical procedure to evaluate the required size of a stiffener is difficult and tedious for the most simple assumptions and becomes practically impossible for actual conditions. The rigorous investigations made by E. Chwalla and A. Novak (44) proved that a previous, simple proposal by Professor Chwalla (36) gives satisfactory values for design purposes. This proposal was adopted. Translated into the nomenclature of this paper, it reads:

$$I_s = \frac{(0.1 + 0.02\,N)\,t^3\,d}{\beta_1^4}\,(\beta_1^2 + 0.625) \ldots\ldots\ldots\ldots\ldots (38)$$

but not to exceed $\frac{0.2\,t^3\,d}{\beta_1^4}\,(\beta_1^2 + 0.625)$. In Eq. 38: β_1 = ratio of the theoretically required stiffener spacing l_s to total depth of web (in cases where l_s changes, the average value of the spacing should be used to compute β_1. For values of β_1 less than 0.4, use 0.4 for β_1); and I_s = moment of inertia of stiffener, in inches^4.

The assumptions made in the derivation of the foregoing empirical formula (Eq. 38) for the required moment of inertia limit the ratio β between 0.4 and 2.5. Values of β outside these limits would lead to results which are invalid. The upper limit is of no practical importance since the ratio β is always below 2.5 in plate-girder design. Values of β of less than 0.4 may occur, however, and it is necessary to make provision for these cases. For these values the theory of orthogonally anisotropic plates may be applied (15) (19) (28) (29). A great number of examples of webs were analyzed with an intermediate stiffener spacing giving ratios β of 0.4 and less. It was found that the proposed formula for the intermediate stiffeners gives an ample section if a value of β_1 of 0.4 is used for all cases in which β is less than 0.4. The required size of intermediate stiffeners thus determined is also in agreement with tests made by E. Seydel (15) (19) (28) (29). For equal stiffeners on both sides of the web, the moment of inertia should be taken about the center line of the web. For stiffeners on one side only, the moment of inertia should be taken about the face of the web in contact with the stiffener.

It will be noted that in Eq. 38 the theoretically required spacing l_s is used in the ratio β_1. The reasons for this are simply to accommodate the formula to practical design considerations. Stress conditions require a certain stiffener spacing which may not be fully realized in the actual design. The spacing of intermediate stiffeners may be made considerably smaller than theoretically required to aline them to stringers in the case of floor beams or between floor beam connections in the case of plate girders.

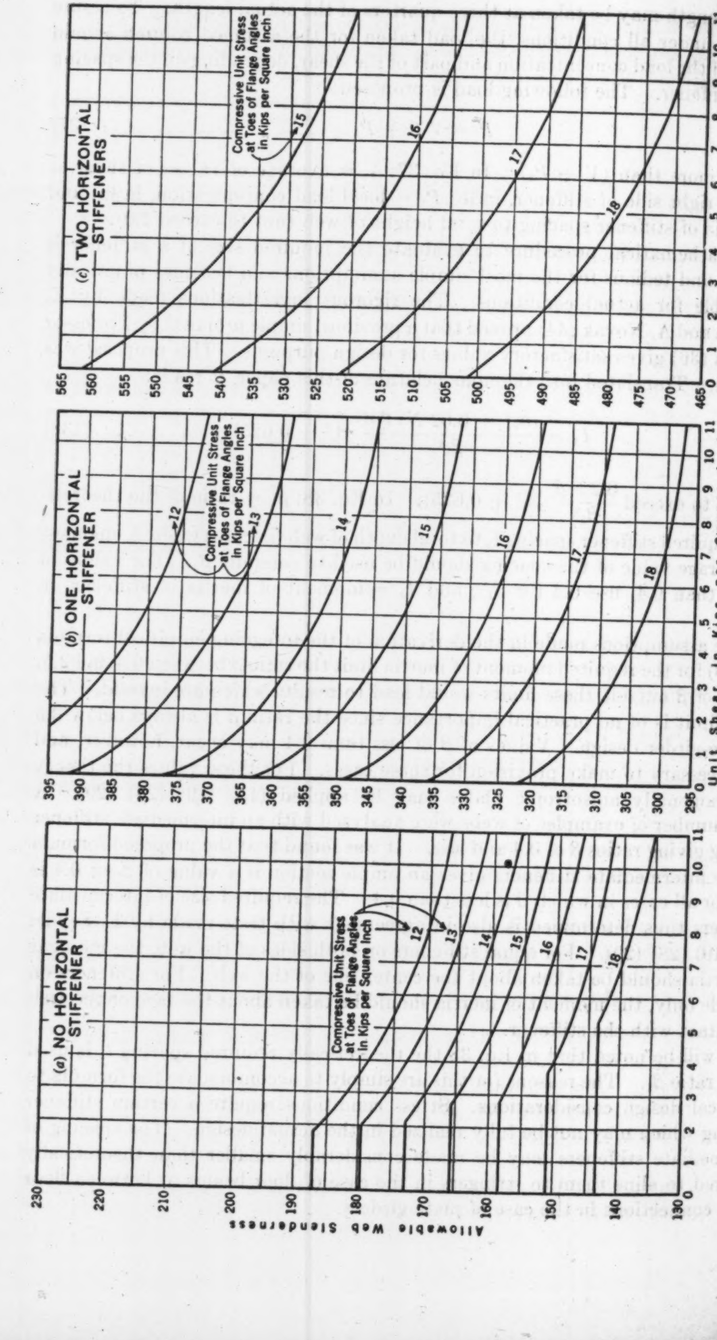


FIG. 2.—ALLOWABLE SLENDERNESS RATIOS $\frac{d}{t}$ FOR CARBON STEEL.

The smaller spacing increases the buckling stability of the web panel above the required limit. Since the formula for the size of the stiffener is balanced with the buckling strength of the web, an equally higher strength in the stiffened web would be attained by using the actual spacing. This is not necessary.

HORIZONTAL STIFFENERS OF WEBS OF PLATE GIRDERS

The general buckling behavior of the web of plate girders subject to load was discussed briefly under the heading of "Design of Webs for Plate Girders," Chapter 3. Buckling of the web is caused by compression or shear, or both. As it is desirable to have a buckling safety factor of 1.5 for compression or shear and one of 1.4 for the combined action of the two, a more detailed study of the behavior of the web becomes necessary.

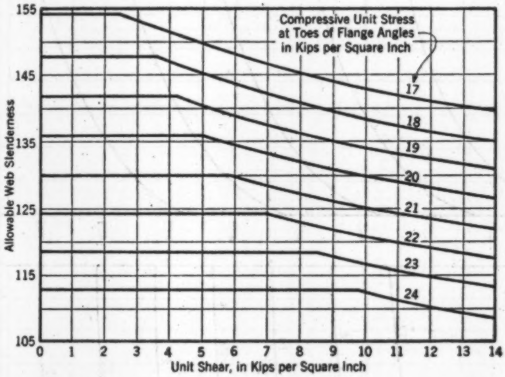


FIG. 3.—ALLOWABLE SLENDERNESS RATIOS $\frac{d}{t}$ FOR SILICON STEEL; NO HORIZONTAL STIFFENERS

To clarify the interdependence of stresses on the buckling stability of the web, the general stability expression, Eq. 1, will first be applied to the unstiffened web of a plate girder.

Carbon Steel with No Horizontal Stiffeners ($N = 0$).—The plate is in pure bending, and $\alpha = 2$ in Eq. 2. Three examples will serve to demonstrate this case.

Example 1.—Maximum compression at toe of flange angles $\sigma = 15{,}000$ lb per sq in. and shear $\tau = 0$. The critical stress is $\sigma_{cr} = 1.5 \times 15{,}000 = 22{,}500$ lb per sq in. By Eq. 1, $22{,}500 = 26{,}210{,}000\, \zeta\, k \left(\frac{t}{d}\right)^2$. For $\alpha = 2$, Eq. 3 gives $k = 24$. For a critical stress of 22,500, Table 3 gives a value of 0.979 for ζ. Introducing these values in Eq. 1, the web slenderness $\frac{d}{t} = 165$.

Example 2.—Maximum compression at toe of flange angles $\sigma = 18{,}000$ lb per sq in. and shear $\tau = 0$. The critical stress now increases to 27,000 lb as in Example 1, the value of coefficient k is again 24, but the value for ζ drops to 0.878. The web slenderness becomes, similarly, $\frac{d}{t} = 143$. To study the effect

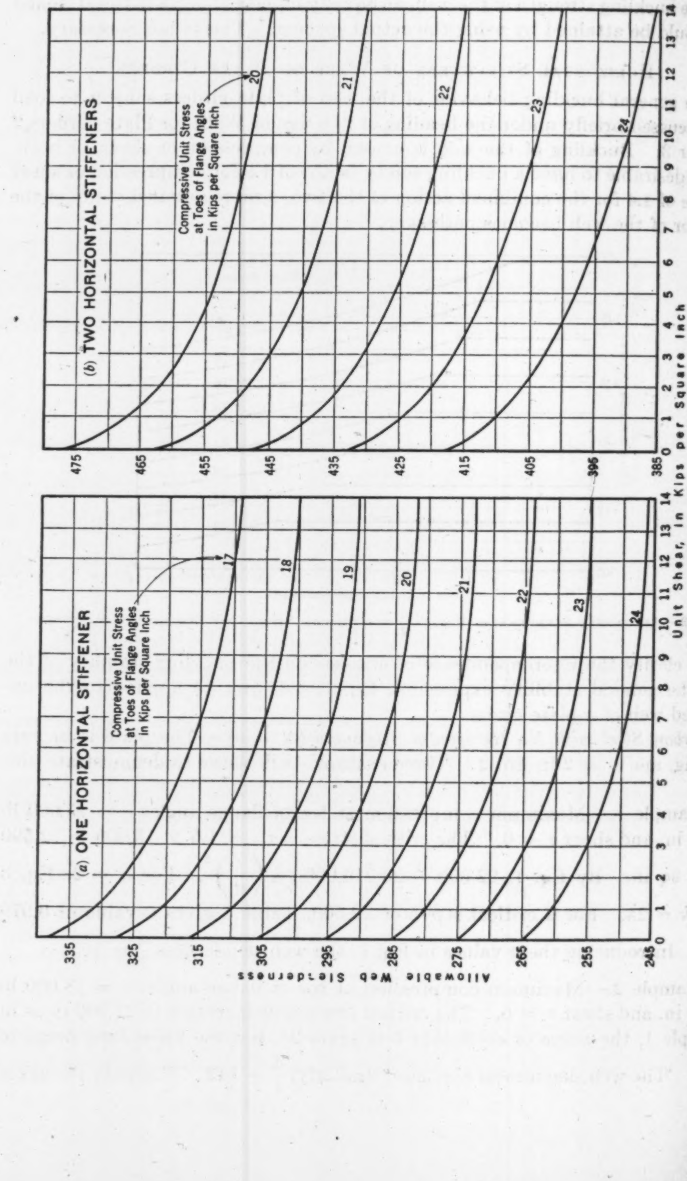


FIG. 4.—ALLOWABLE SLENDERNESS RATIOS $\frac{d}{t}$ FOR SILICON STEEL

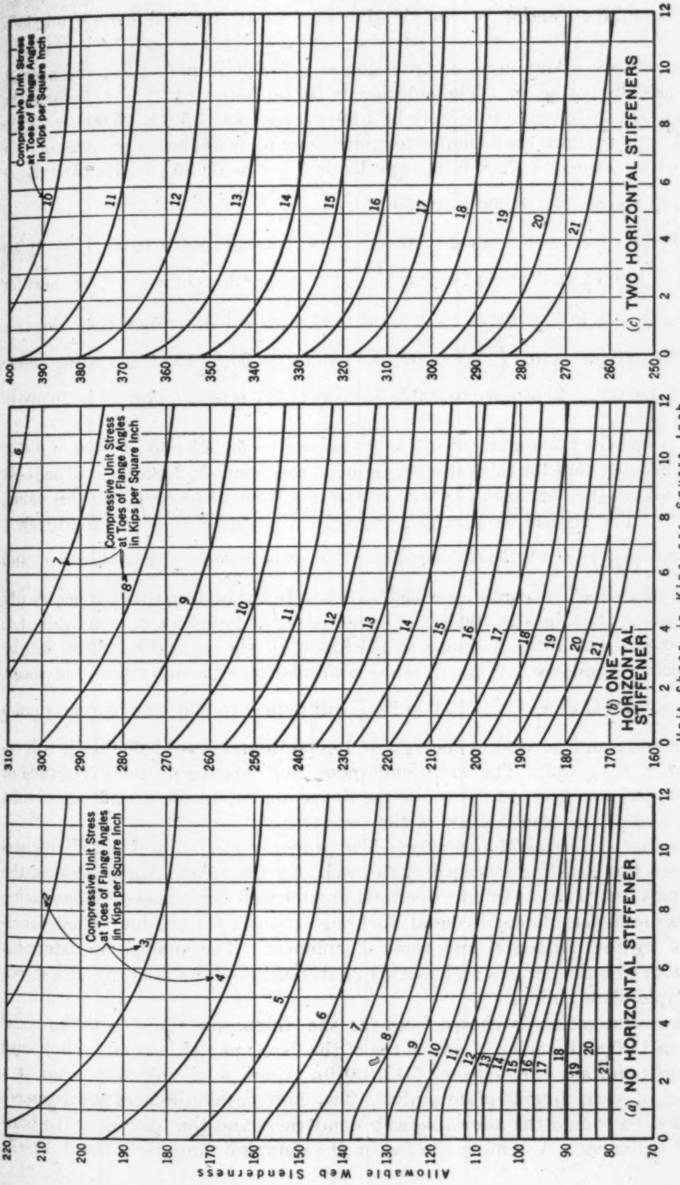


FIG. 5.—ALLOWABLE SLENDERNESS RATIOS $\frac{d}{t}$ FOR ALUMINUM ALLOYS

of the combined action of compression and shear, the following example is given:

Example 3.—Maximum compression $\sigma = 15{,}000$ lb per sq in., and average $\tau = 5{,}000$ lb per sq in. The solution must be found by trial. Assume the factor of safety for compression to be 1.59; σ_{cr} becomes $1.59 \times 15{,}000 = 23{,}850$ lb. The unreduced coefficient k for pure bending is 24, and the value of ζ for the critical stress of 23,850 lb from Table 3 is $\zeta = 0.959$ (see Eq. 5). Eq. 1 reads, $23{,}850 = 26{,}210{,}000 \times 0.959 \times 24 \left(\frac{t}{d}\right)^2$, or $d = 159\,t$.

For the spacing of vertical stiffeners it has been proposed to use $l_s = 10{,}500 \times \frac{t}{\sqrt{\tau}} = 148.5\,t$; the ratio β becomes $\frac{148.5}{159} = 0.93$; the coefficient k for shear for this value of β is (see Table 1) $k_s = 10.1$; the critical shearing stress (Eq. 1) is $\tau_{cr} = 26{,}210{,}000\,\zeta\, 10.3 \left(\frac{t}{d}\right)^2$; and, the value of d being 159 t, the critical shear $\tau_{cr} = 10{,}490\,\zeta$. Reference to Table 8 shows that ζ is unity, and $\tau_{cr} = 10{,}490$ lb per sq in.

The ratio of the actual shear τ to its critical value is $5{,}000 : 10{,}490 = 0.477$. Table 2 shows that for this ratio the reduced coefficient k_d is 85.4%. The shear thus reduces the coefficient k for axial stresses from 24 to 85.4% of this value, or 20.5. The critical axial stress, therefore, is reduced to $\sigma_{cr} = 26{,}210{,}000\,\zeta \times 20.5 \left(\frac{1}{159}\right)^2 = 21{,}250\,\zeta$. A value of ζ must be assumed by trial until σ_{cr} and $21{,}250\,\zeta$ balance in accordance with Table 3. It will be seen that ζ is practically unity and σ_{cr} becomes 21,100 lb. The factor of safety is 1.41, or practically 1.4, as desired. The ratio of depth to thickness, or the web slenderness, can be established similarly for any desired combination of axial stress and shear. For $\sigma = 15{,}000$ lb and $\tau = 11{,}000$ lb, $\frac{d}{t}$ will reduce to 150; and, for maximum compression and maximum shear, $\sigma = 18{,}000$ lb and $\tau = 11{,}000$ lb, it will reduce further to 135. The web slenderness thus depends on the magnitude of the compressive stress at the toe of the flange angles, on the magnitude of unit shear, and on the combination of the two stresses.

Spacing of Horizontal Stiffeners.—The proper placing of horizontal stiffeners increases the buckling strength of the web; for the same critical stresses, the presence of horizontal stiffeners permits greater web slenderness. The spacing of these stiffeners demands equal buckling strength for the individual panels formed by the horizontal and vertical stiffeners. The analysis to determine allowable web slenderness and spacing of stiffeners follows the examples shown for simple plate girders.

An investigation is shown here for the aluminum alloy[3] with an axial stress of 17,000 lb per sq in. at the toe of the flange angles in combination with the maximum allowable shear of 12,500 lb. Such a loading may occur at a support of a continuous plate girder. Two horizontal stiffeners are assumed. It is desired to find the allowable web slenderness and the spacing of the horizontal stiffeners. Assuming the factor of safety for compression as 1.55, the critical stress at the compression toes of the flange angles becomes 1.55 × 17,000 = 26,350 lb per sq in. Let it be further assumed that—if $c_1\,d$ and $c_2\,d$ are the distances, respectively, of the first and second stiffeners to the toe of the compression flange angle—$c_1 = 0.129$ and $c_2 = 0.285$. With these assumptions the buckling stability of the individual panels can be determined. The assumptions must be changed until the computations show a balanced buckling strength.

The spacing of vertical stiffeners is derived from $l_s = 6{,}500 \dfrac{t}{\sqrt{\tau}} = 58.138\,t$ (approximately 58 t). The assumed horizontal stiffener spacing gives the compressive stresses at their location. They are as follows:

Location	Compressive stress, in pounds per square inch
At the toe of flange angles	= 17,000
At first stiffener	= 12,610
At second stiffener	= 7,310

All values for computing α in Eq. 2 are thus known. For the three panels: $\alpha_1 = 0.258$; $\alpha_2 = 0.421$; and $\alpha_3 = 3.326$. Introducing these values in Eq. 3: $k_1 = 4.217$; $k_2 = 4.605$; and $k_3 = 73.959$. The coefficients k are the unreduced values for compression. Applying Eq. 1 to the first panel,

$$26{,}350 = 4.217 \times 9{,}507{,}000\,\zeta \times \left(\frac{t}{c_1\,d}\right)^2 \ldots\ldots\ldots\ldots\ldots (39)$$

The value of ζ from Table 3 is 0.965 for the critical stress of 26,350 lb; from Eq. 39 the depth becomes $d = 297.05\,t$. The depths of the individual panels are $d_1 = 38.32\ t$, $d_2 = 46.34\ t$, and $d_3 = 212.39\ t$. Since the length of each panel is 58.138 t, the length-to-depth ratio becomes $\beta_1 = 1.52$, $\beta_2 = 1.26$, and $\beta_3 = 0.273$. With these values of β, Table 1 yields: $k_1 = 7.08$, $k_2 = 7.88$, and $k_3 = 85.87$. Introduced in Eq. 1, the critical shear for each panel is $\tau_{1cr} = 45{,}840\,\zeta_1$, $\tau_{2cr} = 34{,}890\,\zeta_2$, and $\tau_{3cr} = 18{,}100\,\zeta_3$.

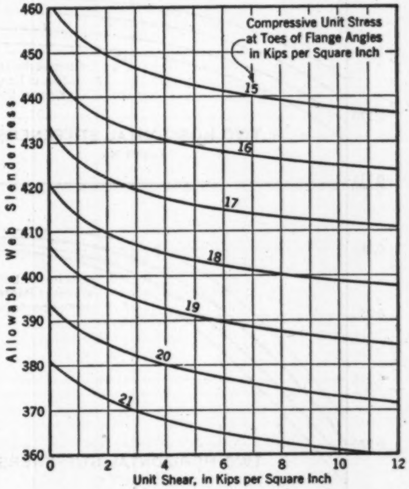


FIG. 6.—ALLOWABLE SLENDERNESS RATIOS $\frac{d}{t}$ FOR ALUMINUM ALLOY, WITH THREE HORIZONTAL STIFFENERS

Table 8 gives the values of ζ by balancing both sides of the foregoing expressions for the critical shearing stress. This balancing is done by trial, as follows:

The second expression reads $\tau_{2cr} = 34{,}890\,\zeta_2$. As a first trial, $\zeta_2 = 0.80$, or $\tau_{2cr} = 34{,}890 \times 0.80 = 27{,}910$. For $\tau_{cr} = 27{,}000$, Table 8 gives a value of $\zeta = 0.898$, and, for $\tau_{cr} = 28{,}000$, $\zeta = 0.804$. The assumed value for ζ_2 of 0.80, giving a critical stress of 27,910 lb, is thus closely correct. The balanced values for the modulus coefficients are: $\zeta_1 = 0.63$, $\zeta_2 = 0.80$, and $\zeta_3 = 1.0$. The critical stresses, therefore, are $\tau_{1cr} = 28{,}880$ lb, $\tau_{2cr} = 27{,}910$ lb, and $\tau_{3cr} = 18{,}100$ lb. The ratio of the actual shear to its critical value becomes 0.43, 0.45, and 0.69, respectively. The reduction in the stability coefficient k

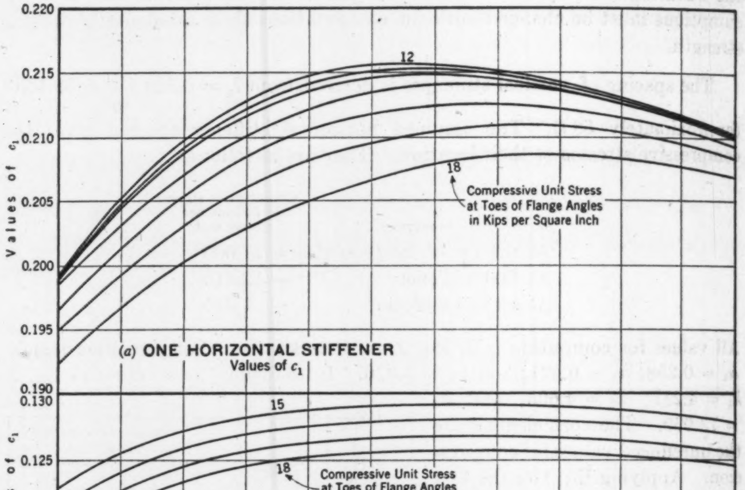


FIG. 7.—SPACING OF HORIZONTAL STIFFENERS; CARBON STEEL

for bending stresses due to the simultaneous action of shear becomes (see Table 2): $k_{1d} = 0.88\,k_1$, $k_{2d} = 0.87\,k_2$, and $k_{3d} = 0.68\,k_3$. Introducing, for k_1, k_2, and k_3, the computed values of 4.217, 4.605, and 73.959, respectively, the reduced coefficients are $k_{1d} = 3.71$, $k_{2d} = 4.01$, and $k_{3d} = 50.29$. The shear, thus, reduces the critical compressive stress to: $\sigma_{1cr} = 24{,}100\,\zeta_1$, $\sigma_{2cr} = 17{,}800\,\zeta_2$, and $\sigma_{3cr} = 10{,}600\,\zeta_3$. These expressions must be balanced with the aid of

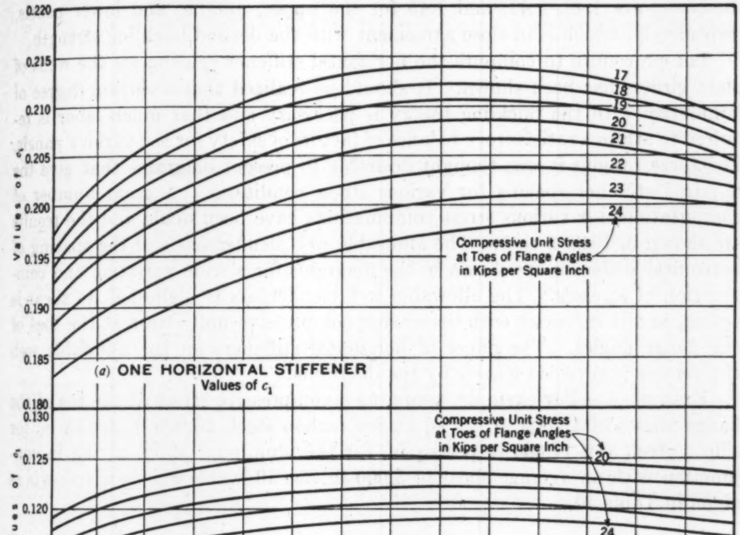


FIG. 8.—SPACING OF HORIZONTAL STIFFENERS; SILICON STEEL

Table 3, similarly to the foregoing balancing of shears: ζ_1 is 0.99, and ζ_2 and ζ_3 are unity. The critical stresses thus become 23,840, 17,800, and 10,600 lb per sq in.; and the corresponding actual stresses are 17,000, 12,610, and 7,310 lb per sq in., as originally assumed. The factors of safety against buckling, therefore, are 1.40, 1.41, and 1.45 for the upper, middle, and lower panels, respectively, which is in close agreement with the desired buckling strength.

The procedure to compute the horizontal stiffener spacing for the webs of plate girders has been shown. It should be realized that a certain degree of familiarity with the buckling theory is necessary and that much labor is required to attain a satisfactory balance of factors of safety for the various panels. For these reasons it was thought desirable to present diagrams that give the required stiffener spacing for various stress conditions. A great number of computations for various stress combinations have been made and the results are shown in Figs. 2 to 9. The allowable web slenderness and the spacing of horizontal stiffeners are shown in the diagrams for a wide variation and combination of stresses. The allowable web slenderness is highest if no shear is acting, and it increases with decreasing compressive unit stress at the toes of the flange angles. The effect of horizontal stiffeners on the allowable web slenderness is revealed clearly by the diagrams.

Example 4.—For example, assuming a compressive stress at the toe of the flange angles of 15,000 lb per sq in. for carbon steel, 20,000 lb per sq in. for silicon steel, and 17,000 lb per sq in. for the aluminum alloy, acting in conjunction with an average shear of 5,000 lb, the allowable web slenderness is as given in Table 10.

TABLE 10.—ALLOWABLE WEB SLENDERNESS, $\frac{d}{t}$, FOR $\tau = 5{,}000$ LB PER SQ IN.

Material	Assumed stress, σ, in pounds per square inch	NUMBER OF STIFFENERS, N			
		0	1	2	3
Carbon steel..........	15,000	159	340	534	....
Silicon steel..........	20,000	136	288	450	....
Aluminum alloy.......	17,000	90	191	301	416

Structural engineers are acquainted with the fact that in present plate-girder designs the unit shear is low in most cases, and the thickness of the web is commonly governed by the allowable web slenderness. The proper placing of horizontal stiffeners increases the allowable web slenderness materially and will effect considerable economy.

Example 5.—For example, assume that a four-lane highway bridge has a simple span of 120 ft. Plate girders are selected for the two main girders. The total depth of each girder is 138.5 in., back to back of 8-in. angles, resulting in a net depth of $d = 122.5$ in. between toes of flange angles. Carbon steel is used for the girders, and it is assumed that the highest compressive unit stress at the toes of the flange angles is 15,000 lb per sq in. and that this stress prevails not only at midspan, but also near the quarter point where the total shear is taken as 250,000 lb. Therefore, the latter section controls the design.

The plate girder is first proportioned with no horizontal stiffeners. The web thickness is assumed at $t = \frac{3}{4}$ in., resulting (at the controlling section) in a unit shear of 2,400 lb. The allowable web slenderness, from Fig. 2(*a*), for $\sigma = 15{,}000$ and $\tau = 2{,}400$ lb, is 165, which conforms with the assumed value of $\frac{122.5}{0.75} = 163.3$.

With one horizontal stiffener the assumed web thickness is reduced from $\frac{3}{4}$ in. to $\frac{3}{8}$ in., with a corresponding increase in unit shear to 4,800 lb per sq in. The actual web slenderness is $\frac{122.5}{0.375} = 327$. The allowable value, from Fig. 2(*b*), for $\sigma = 15{,}000$ and $\tau = 4{,}800$, is 341; the allowable web slenderness is not even fully utilized, although the web thickness was reduced to one half. The substantial economy effected by horizontal stiffeners is evident. The spacing of the stiffeners is taken from Fig. 7(*a*); it is $0.2134 \times d = 26$ in. from the toes of the upper flange angles.

A study of Figs. 2 to 9 will show the identical behavior of the three materials investigated. The curves for web slenderness show a marked similarity. The required spacing for the horizontal stiffeners is practically the same for carbon steel and the aluminum alloy, whereas, for silicon steel the spacing deviates only slightly. The reason for this behavior is due to identical values of ζ for carbon steel and the aluminum alloy studied. For silicon steel the corresponding values of ζ are somewhat lower because the higher allowable unit stress brings the critical stress nearer to the plastic range.

Size of Horizontal Stiffeners.—To determine the size of horizontal stiffeners of plate girders Eq. 19 must be applied to the problem. The solution follows the pattern of the determination of longitudinal stiffeners in columns, except that the analysis becomes much more involved. The equivalent coefficient k is no longer a multiple of k_0 depending on the number of stiffeners, as in the case of columns. Its magnitude depends on the web slenderness and the critical buckling stress, and these, in turn, are functions of the actual stress conditions. This is illustrated by a simple example of a carbon-steel plate girder with one horizontal stiffener.

Example 6.—It is assumed that the maximum compression at the toes of flange angles is 15,000 lb per sq in. and that no shear is acting. For this condition a factor of safety of 1.5 was taken, which results in a critical stress of 22,500 lb and a value of $\zeta = 0.979$, from Table 3. For the web slenderness Fig. 2(*b*) gives $\frac{d}{t} = 363$. Introducing these values in Eq. 1 and solving for coefficient k,

$$k = \frac{\sigma_{cr}}{26{,}210{,}000\,\zeta}\left(\frac{d}{t}\right)^2 = \frac{22{,}500}{26{,}210{,}000 \times 0.979}(363)^2 = 116$$

Example 7.—The maximum compression at the toe of the flange angles is 18,000 lb per sq in., and the shear is 11,000 lb per sq in. For this stress condi-

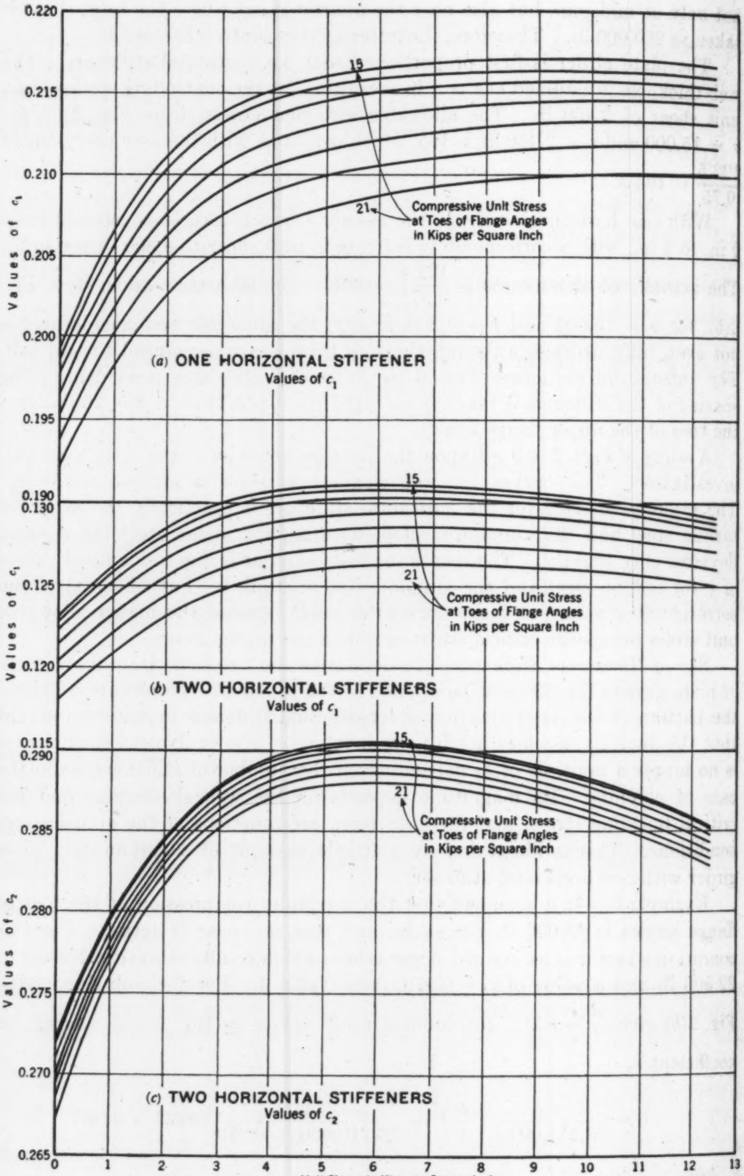
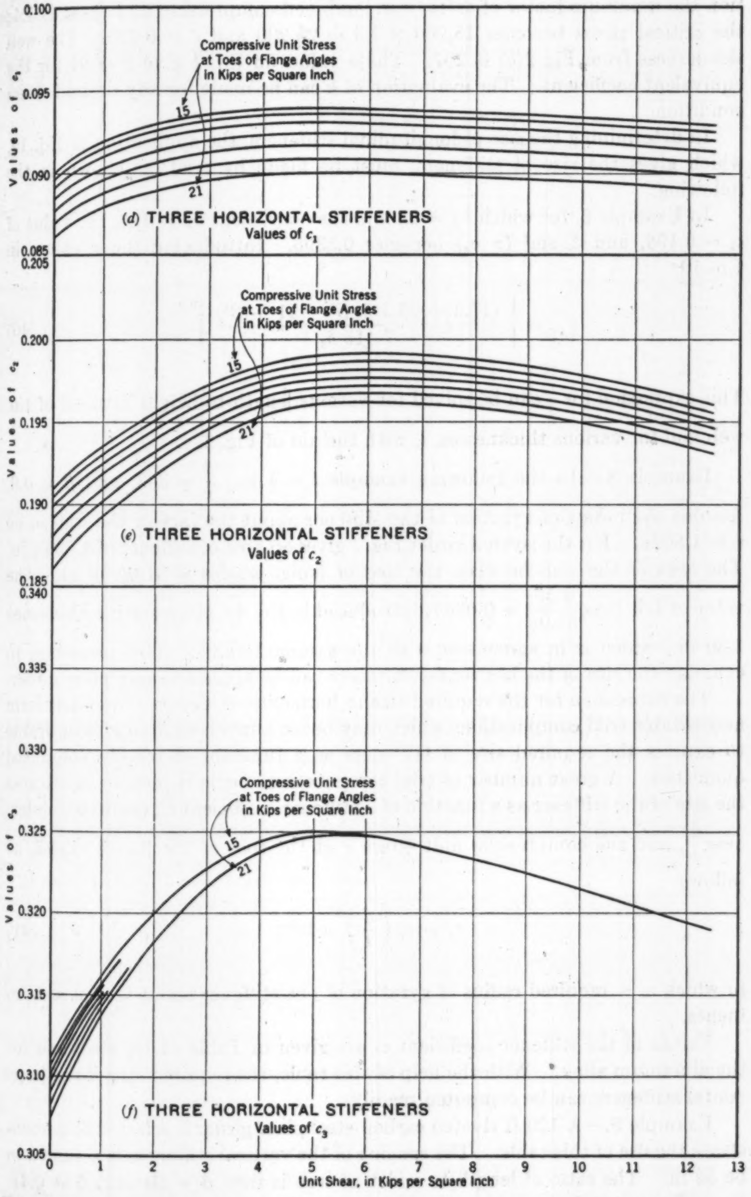


FIG. 9.—SPACING OF HORIZONTAL



(d) THREE HORIZONTAL STIFFENERS
Values of c_1

(e) THREE HORIZONTAL STIFFENERS
Values of c_2

(f) THREE HORIZONTAL STIFFENERS
Values of c_3

STIFFENERS; ALUMINUM ALLOY

tion the minimum factor of safety for combined compression and shear is 1.4; the critical stress becomes 18,000 × 1.4 = 25,200 and $\zeta = 0.930$. The web slenderness from Fig 2(*b*) is 297. These values in Eq. 1 give $k = 91$ for the equivalent coefficient. The evaluation of k can be made for any desired stress condition.

In determining the size of longitudinal stiffeners, the application of Eq. 19, which gives the size of stiffeners, must be made by trial, as shown in the following:

In Example 6, for which $k_0 = 24$ and the equivalent $k_s = 116$, the value of $c_1 = 0.198$, and $\Sigma \sin^2 (\pi c_s)$ becomes 0.3395. Introducing these values in Eq. 19:

$$\frac{r}{t} = \left[\frac{(17.3 + 13.1\,\delta)\,\beta^2 - (1 + \beta^4)}{7.415\,\delta_s} \right]^{0.5} \qquad (40)$$

This expression for $\frac{r}{t}$ can be solved for various length-to-height ratios β of the web and for various thicknesses, t, with the aid of Fig. 1.

Example 8.—In the following example $t = \frac{3}{8}$ in., $\frac{d}{t} = 363$, and $\beta = 0.6$. Assume the radius of gyration of the stiffener about the face of the web to be $r = 1.80$ in. For the riveted girder Fig. 1 gives an area of stiffener of 1.55 sq in. The area of the web between the toes of flange angles is 51.05 sq in. The value of δ_s is thus $\frac{1.55}{51.05} = 0.0304$; introduced in Eq. 40, the resulting r becomes 1.80 in., which is in agreement with the assumed value. This procedure to evaluate the size of the horizontal stiffeners can be applied to any plate girder.

The expression for the required size of horizontal stiffeners in implicit form necessitates trial computations which may become involved, and it is desirable to express the required size of stiffeners as a function of readily computed quantities. A great number of trial computations made it possible to express the size of the stiffener as a function of a stiffener coefficient c_r, the web slenderness $\frac{d}{t}$, and the compressive unit stress σ at the toes of the flange angles, as follows:

$$r_1 = c_r \left(\frac{d}{t}\right)^2 \sigma\, 10^{-6} \qquad (41)$$

in which r_1 = required radius of gyration of one stiffener about face of web, in inches.

Values of the stiffener coefficient c_r are given in Table 11 for steel and for the aluminum alloy. With the help of this table, the required size of the horizontal stiffeners can be computed readily.

Example 9.—A 120-ft riveted carbon-steel plate girder is selected to demonstrate the use of this table. The spacing of the vertical stiffeners is assumed to be 50 in. The ratio of length to width of web is then $\beta = 50 : 122.5 = 0.41$. From Table 11(*a*) the value of c_r is found by interpolation to be 0.69 for the web thickness of $\frac{3}{8}$ in. With $\sigma = 15$ kips per sq in. and $\frac{d}{t} = 327$, the required radius of gyration about the face of the web becomes $r = 0.69 \times (327)^2 \times 15 \times 10^{-6} = 1.11$ in. The smallest angle which permits riveting is $3\frac{1}{2}$

TABLE 11.—VALUES OF STIFFENER COEFFICIENT, c_r

Values of β	VALUES OF c_r FOR THE FOLLOWING VALUES OF WEB THICKNESS, t, IN INCHES:											
	5/16	3/8	7/16	1/2	9/16	5/8	5/16	3/8	7/16	1/2	9/16	5/8
	(*a*) RIVETED STEEL; ONE STIFFENER						(*b*) WELDED STEEL; ONE STIFFENER					
0.2	0.14	0.25	0.37	0.49	0.61	0.74	0.28	0.38	0.48	0.59	0.70	0.81
0.3	0.35	0.48	0.62	0.77	0.91	1.06	0.45	0.58	0.72	0.85	1.00	1.14
0.4	0.51	0.67	0.84	1.01	1.18	1.36	0.60	0.76	0.93	1.10	1.30	1.46
0.5	0.64	0.83	1.02	1.22	1.43	1.64	0.72	0.92	1.12	1.33	1.54	1.76
0.6	0.75	0.97	1.19	1.42	1.66	1.90	0.83	1.06	1.30	1.54	1.79	2.04
0.7	0.85	1.09	1.34	1.59	1.85	2.12	0.92	1.18	1.44	1.72	1.99	2.28
0.8	0.93	1.19	1.47	1.74	2.03	2.32	1.00	1.28	1.57	1.87	2.18	2.49
0.9	1.00	1.28	1.57	1.87	2.18	2.49	1.07	1.37	1.69	2.01	2.33	2.67
1.0	1.07	1.37	1.67	1.99	2.29	2.64	1.14	1.46	1.79	2.13	2.48	2.84
	(*c*) RIVETED STEEL; TWO STIFFENERS						(*d*) WELDED STEEL; TWO STIFFENERS					
0.2	0.16	0.22	0.29	0.36	0.43	0.50	0.20	0.27	0.33	0.40	0.48	0.55
0.3	0.23	0.31	0.38	0.46	0.55	0.63	0.27	0.35	0.43	0.52	0.60	0.69
0.4	0.29	0.38	0.47	0.56	0.66	0.76	0.34	0.43	0.52	0.62	0.72	0.82
0.5	0.35	0.45	0.55	0.66	0.77	0.88	0.40	0.50	0.61	0.72	0.83	0.95
0.6	0.41	0.52	0.63	0.75	0.87	0.99	0.46	0.58	0.70	0.82	0.95	1.08
0.7	0.46	0.58	0.70	0.83	0.96	1.09	0.51	0.64	0.77	0.91	1.05	1.19
0.8	0.51	0.64	0.77	0.90	1.04	1.18	0.56	0.70	0.84	0.98	1.13	1.28
0.9	0.55	0.68	0.82	0.97	1.11	1.26	0.61	0.75	0.90	1.05	1.21	1.37
1.0	0.59	0.73	0.87	1.02	1.17	1.33	0.65	0.80	0.95	1.11	1.27	1.44
	(*e*) RIVETED ALUMINUM ALLOY; ONE STIFFENER						(*f*) RIVETED ALUMINUM ALLOY; TWO STIFFENERS					
0.2	0.26	0.34	0.43	0.52	0.61	0.71	0.30	0.40	0.51	0.61	0.72	0.84
0.3	0.73	0.95	1.19	1.45	1.72	2.00	0.49	0.65	0.82	0.99	1.16	1.33
0.4	1.09	1.43	1.78	2.15	2.54	2.94	0.65	0.86	1.06	1.27	1.48	1.69
0.5	1.40	1.83	2.26	2.71	3.16	3.63	0.79	1.03	1.27	1.50	1.74	1.97
0.6	1.67	2.16	2.65	3.16	3.66	4.19	0.91	1.18	1.45	1.70	1.96	2.22
0.7	1.90	2.42	2.99	3.53	4.08	4.66	1.03	1.32	1.61	1.89	2.17	2.43
0.8	2.09	2.68	3.29	3.89	4.48	5.11	1.13	1.44	1.75	2.06	2.35	2.63
0.9	2.27	2.92	3.58	4.24	4.86	5.51	1.23	1.56	1.89	2.22	2.51	2.81
1.0	2.45	3.15	3.85	4.55	5.22	5.90	1.31	1.66	2.01	2.36	2.67	2.98
	(*g*) RIVETED ALUMINUM ALLOY; THREE STIFFENERS											
0.2	0.23	0.30	0.38	0.46	0.54	0.63	....	....	....	....	....	....
0.3	0.34	0.45	0.55	0.65	0.77	0.88	....	....	....	....	....	....
0.4	0.44	0.56	0.69	0.82	0.95	1.08	....	....	....	....	....	....
0.5	0.52	0.66	0.81	0.96	1.10	1.24	....	....	....	....	....	....
0.6	0.60	0.75	0.91	1.08	1.23	1.39	....	....	....	....	....	....
0.7	0.66	0.83	1.01	1.18	1.35	1.51	....	....	....	....	....	....
0.8	0.72	0.91	1.10	1.28	1.46	1.63	....	....	....	....	....	....

$\times 3 \times \frac{5}{16}$, with the $3\frac{1}{2}$ in. leg facing the web. The radius of gyration of this angle about the face of web is 1.21 in., which is in excess of the required value.

SUMMARY

The paper discusses the basic laws governing the buckling of metal elements which form structural members. The behavior of these elements, subjected to various stress conditions and combinations, is analyzed. The results are applied to unstiffened plates of columns and girders. The studies are then extended to structural parts reinforced by stiffeners. The analysis leads to general expressions which enable the engineer to judge and weigh the action and behavior of stiffened plates. These expressions are applied to column plates and webs of plate girders reinforced by stiffeners. An extensive study of the behavior of stiffened plates in columns and girder made it possible to establish simple rules for practical design. Tables and diagrams have been prepared from them to facilitate the work.

CONCLUSIONS

The study of the elastic stability of metal elements given in the paper has clarified the general behavior of buckling. It has made possible the establishment of simple rules for the design of structural members reinforced by stiffeners. The application of these rules results in more dependable and more economical structures. It gives greater freedom to the structural application of metals and will lead to simpler fabrication and erection.

ACKNOWLEDGMENT

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APPENDIX

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(50) **"Über die Anwendung der energetischen Methode auf Stabilitätsprobleme,"** by K. Marguerre, *Jahrbuch der deutschen Luftfahrtforschung*, Berlin, 1938, p. 433.

(51) **"Die Knickung der rechteckigen Platte durch Schubkräfte,"** by S. Iguchi, *Ingenieur-Archiv*, Vol. 9, Berlin, 1938, p. 1; Vol. 10, Berlin, 1939, p. 77.

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DISCUSSION

Louis Balog,[5] **Esq.** (by letter).—For the solution of the general stability equation (Eq. 1), the knowledge of the stability coefficient k and the variation of the modulus of elasticity expressed by the modulus factor ζ are necessary. Stability coefficient k is a function of the dimensions, boundary, and loading conditions of the plate. The values of this coefficient are independent of the material constants E and m. The determination of the k-values for the various conditions of support and loading is a mathematical procedure that can be solved without a knowledge of the characteristics of the plate material.

For simply supported rectangular plates subject to direct stresses along the depth of the plate, Eq. 3 gives the approximate minimum values of coefficient k. In Table 1 the k-values for uniform shear along the four edges of the plate are given. Table 2 gives the reduction of the k-values of pure bending by the simultaneous action of shear.

A frequently occurring design case is that involving bending, axial stress, and shear. For the most economical design, the distribution of the material in sections composed of shapes should be such that bending and axial stresses produce stresses equal and of the opposite sign in the extreme fibers of the sections. This assures that the flange areas are minimum and the allowable web slenderness is maximum. The k-values for pure bending, which are the largest and which will be reduced the least by shear, apply for the web design.

It is not always practicable, however, to satisfy the criteria of the most economical design. Constant sections and rolled beams also often occur as members subject to bending, axial stress, and shear. In these cases k will be smaller than for pure bending, and the reduction of k due to shear will be larger than that given in Table 2. The authors' diagrams for the allowable slenderness ratios cannot be used.

The slenderness-ratio diagrams are based on the minimum value of k for pure bending in simply supported plates, with the corresponding shear reduction. If the width-to-depth ratio of the plate is smaller than the β-ratio to which the minimum k-value corresponds, k increases. In the stress-distribution range between pure bending and uniform compression, k decreases and the reduction of k due to the simultaneous action of shear increases. Safety, uniformity of the safety factors of the parts of the structure, and economical considerations require that the allowable plate slenderness shall be determined with due consideration of the width-to-depth ratio β and the actual stress distribution wherever these are significantly different from those on which the allowable slenderness ratio diagrams were based.

In structures, the support condition of the plates is seldom clearly defined. Deviations from the true plane plate surface may more than offset the advantages of restraint. It is prudent, therefore, to use, for most structural plates, the buckling coefficients of simply supported plates.

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[5] Engr. with Leon S. Moisseiff, Cons. Engr., New York, N. Y.

The computation of the rigorous k-values is a laborious operation. Simple relations between the ratio β and the loading conditions, giving sufficiently close approximations for the buckling coefficient k, are of interest only in practical design. Such equations for the stress-variation range from uniform compression to pure bending may be given as follows:

Let σ_1 be the compressive stress at the toe of the top flange angle and σ_2 the compressive or tensile stress at the toe of the bottom flange angle, and the ratio $\frac{\sigma_2}{\sigma_1}$ will express linear stress variation in the depth of the plate. Then the basic loading conditions of the plate, uniform compression, triangular stress distribution, and pure bending are expressed by: $\frac{\sigma_2}{\sigma_1} = 1$; $\frac{\sigma_2}{\sigma_1} = 0$; and $\frac{\sigma_2}{\sigma_1} = -1$, respectively. It is noted that $\frac{\sigma_2}{\sigma_1} = 1 - \alpha$, with α defined by Eq. 2.

The k-values for the case of $\frac{\sigma_2}{\sigma_1} = 1$ or $\alpha = 0$ (uniform compression) can be obtained from the simple equation, when $\beta \leqq \sqrt{2}$,

$$k = \left(\beta + \frac{1}{\beta}\right)^2 = k_u \qquad (42)$$

For the case in which $\frac{\sigma_2}{\sigma_1} = 0$ or $\alpha = 1$ (triangular stress distribution), when $\beta \leqq 1$,

$$k = 1.91\, k_u = k_t \qquad (43)$$

When $\beta > 1$, the coefficient of $k_t = 7.64$. For the stress variation range between $\frac{\sigma_2}{\sigma_1} = 1$ (uniform compression) and $\frac{\sigma_2}{\sigma_1} = 0$ (triangular stress distribution), Professor Chwalla proposes:[6]

When $\beta \leqq 1$

$$k = k_u \frac{2.1}{1.1 + \frac{\sigma_2}{\sigma_1}} \qquad (44a)$$

and, for $\beta > 1$,

$$k = \frac{8.4}{1.1 + \frac{\sigma_2}{\sigma_1}} \qquad (44b)$$

For $\frac{\sigma_2}{\sigma_1} = -1$ or $\alpha = 2$ (pure bending), when $\beta \leqq 0.67$,

$$k = 15.870 + \frac{1.87}{\beta^2} + 8.6\, \beta^2 = k_b \qquad (45)$$

When $\beta > 0.67$, the coefficient $k_b = 23.9$. For the stress variation range between $\frac{\sigma_2}{\sigma_1} = 0$ (triangular stress distribution) and $\frac{\sigma_2}{\sigma_1} = -1$ (pure bending), the German Railroads specify[7]

$$k = \left(1 + \frac{\sigma_2}{\sigma_1}\right) k_t - \frac{\sigma_2}{\sigma_1} k_b + 10 \frac{\sigma_2}{\sigma_1}\left(1 + \frac{\sigma_2}{\sigma_1}\right) \qquad (46a)$$

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[6] "Erläuterungen zum Normblattentwurf DIN E 4114," Stahlbau-Verband, Berlin, 1939, p. 9.

[7] "Berechnungsgrundlagen für stählerne Eisenbahnbrücken," Berlin, 1937, p. 60.

or, if the stress variation is expressed by α,

$$k = (2 - \alpha)\,k_t - (1 - \alpha)\,k_b + 20 - 30\,\alpha + 10\,\alpha^2 \ldots\ldots\ldots (46b)$$

The k-values obtained by means of the foregoing simple equations will indicate whether or not the slenderness ratio diagrams of the authors can be used in design cases different from pure bending. The coefficients apply when the direct stresses are not accompanied by shear. Shearing stresses reduce the stability of the plate submitted to direct stresses. This reduction is the smallest for pure bending and the largest for uniform compression.

It is of practical importance, for design purposes, to establish a sufficiently accurate and simple functional relation between the interdependent critical stress combinations σ_c and τ_c. Such relations can be derived from the following considerations: If one of the stresses in the combination is large the other must be small. If the plate is subject only to direct stresses, $\sigma_{cr} = \sigma_c$; and, if only shearing stresses act, $\tau_{cr} = \tau_c$. The ratios $\frac{\sigma_c}{\sigma_{cr}}$ and $\frac{\tau_c}{\tau_{cr}}$ may have any value between zero and one. This can be expressed by a general equation of the form

$$\frac{\sigma_c}{\sigma_{cr}} = f\left(\frac{\tau_c}{\tau_{cr}}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (47)$$

in which the relation changes with β.

Plotting $\frac{\sigma_c}{\sigma_{cr}}$ against $\frac{\tau_c}{\tau_{cr}}$, curves are obtained intersecting both coordinate axes at unity. For pure bending and shear, the curve should be perpendicular at, and symmetrical about, both axes because the reversal of the sign of either moment or shear does not change the character of the stress condition of the plate. For uniform compression and shear, the change of the sign of the shear does not alter the stress condition; hence, the curve should be perpendicular at and symmetrical about the $\frac{\sigma_c}{\sigma_{cr}}$-axis. This curve, however, cannot be symmetrical about, nor can it have a rectangular intersection with, the $\frac{\tau_c}{\tau_{cr}}$-axis because reversal of the sign of the direct stresses changes the stress condition of the plate. These considerations suggest, for the approximation of the critical stress combinations, the following power relation:

$$\frac{\sigma_c}{\sigma_{cr}} = 1 - \left(\frac{\tau_c}{\tau_{cr}}\right)^n \ldots\ldots\ldots\ldots\ldots\ldots\ldots (48)$$

The combined axial compression and torsion tests made with a great number and variety of thin-walled cylinders by F. J. Bridget, C. C. Jerome, and A. B. Vosseller[8] give 2.63 for the average value of n. The results of these tests and the results of the theoretical investigations for combined uniform compression and shear, made by Southwell and Skan (7), Wagner (11), Schmieden (34), Wansleben (35), Chwalla (40), and Iguchi (51), are all covered sufficiently well for design purposes by the general empirical relation Eq. 48, if the value of n is properly selected. The value of exponent n is a function of β.

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[8] "Some New Experiments on Buckling of Thin-Wall Construction," by F. J. Bridget, C. C. Jerome, and A. B. Vosseller, *Transactions*, A. S. M. E., Vol. 56, 1934, pp. 569–578.

For simply supported rectangular plates under the action of uniform compression and shear, $n = 2$ gives the best approximation of the relation between the stress combinations σ_c and τ_c, for all values of $\beta \leqq 1$; thus:

$$\frac{\sigma_c}{\sigma_{cr}} = 1 - \left(\frac{\tau_c}{\tau_{cr}}\right)^2 \dots\dots\dots\dots (49)$$

and so the percentage of k available for direct stress is

$$k_d = 1 - \left(\frac{\tau}{\tau_{cr}}\right)^2 \dots\dots\dots\dots (50)$$

Interpolating between the values of k_d obtained from Eq. 50 and the values given in Table 2, safe values for k_d will be obtained for the direct-stress variation range between uniform compression and pure bending. These values are given in Table 12, which shows that the reduction percentages k_d increase considerably as compression extends over the larger part of the section.

It is interesting to note that the reduction of bending stresses is similar to the reduction of pure axial tension stresses by the simultaneous action of shear. Tests made by E. A. Davis[9] determine the tension and torsion (shear) stress combinations that caused the failure of cylindrical bars and the ultimate values of the stresses when they were applied separately. Plotting from these tests the ratios $\frac{\sigma_c}{\sigma_u}$ against $\frac{\tau_c}{\tau_u}$, a curve similar to the curve for k_d represented by Table 2 will be obtained. The approximate power relation, Eq. 48, for both cases takes the form

$$\frac{\sigma_c}{\sigma_{cr}} = \frac{\sigma_c}{\sigma_u} = \left[1 - \left(\frac{\tau_c}{\tau_{cr}}\right)^2\right]^{0.5} \dots\dots\dots\dots (51)$$

Eq. 51 or the values in Table 2 can be used for k_d in the stress-variation range between pure bending and uniform tension with a combination of shear.

With the knowledge of the k-values and the plate material constants E and m, the critical buckling stresses can be determined as long as they are smaller than the proportional limit of the plate material. For higher critical stresses, the variation of the modulus of elasticity as expressed by the ζ-values must be considered. The critical stresses in structural design, as a rule, are higher than the proportional limit, and the ζ-values assume similar importance to buckling coefficient k in the design of plates.

The k-values are obtained by analytical methods; the ζ-values can be determined only empirically. In Table 3 the authors give ζ-values for structural carbon and silicon steel, and for aluminum alloy. These ζ-values were derived from curves approximating the critical stresses of tested columns.

The multiplier $\epsilon = \frac{E_z}{E}$ of the Euler strength of columns establishes a functional relation $\epsilon = f(\sigma_{cr})$ between the critical stress and elasticity modulus. The integral curve of this function represents the compressive stress-strain relation, $\sigma_{cr} = f(\mu)$. This stress-strain relation is materially different from the compression stress-strain relation, $\sigma_c = f(\Delta_c)$, which expresses the deformation of a compression-test specimen with increasing stress. The stress-strain curves of steels and aluminum, $\sigma_c = f(\Delta_c)$, as determined by Erich Siebel and Anton Pomp[10] are higher than the tension stress-strain curves, $\sigma_t = f(\Delta_t)$, whereas the curves, $\sigma_{cr} = f(\mu)$, representing the relation of the buckling strains to the critical stress of column specimens of various lengths, are always below the tension stress-strain diagrams.

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[9] "Combined Tension-Torsion Tests on a 0.35 Per Cent Carbon Steel," by E. A. Davis, *Transactions*, A. S. M. E., Vol. 62, October, 1940, p. 577.

TABLE 12.—REDUCTION OF k (EQS. 42 TO 46) DUE TO SIMULTANEOUS ACTION OF SHEAR

$\frac{\sigma_2}{\sigma_1}$	VALUE OF k_s FOR THE RATIOS $\frac{\tau}{\tau_{cr}}$ EQUAL TO:									
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
1.0	0.990	0.960	0.910	0.840	0.750	0.640	0.510	0.360	0.190	0.000
0.9	0.990	0.961	0.912	0.843	0.755	0.646	0.518	0.370	0.200	0.000
0.8	0.990	0.961	0.913	0.846	0.759	0.652	0.526	0.379	0.210	0.000
0.7	0.990	0.962	0.915	0.849	0.764	0.658	0.534	0.389	0.220	0.000
0.6	0.990	0.962	0.916	0.852	0.768	0.664	0.542	0.398	0.230	0.000
0.5	0.990	0.963	0.918	0.855	0.773	0.670	0.550	0.408	0.240	0.000
0.4	0.990	0.963	0.919	0.858	0.777	0.676	0.558	0.417	0.250	0.000
0.3	0.990	0.964	0.921	0.861	0.782	0.682	0.566	0.427	0.260	0.000
0.2	0.990	0.964	0.922	0.864	0.786	0.688	0.574	0.436	0.270	0.000
0.1	0.990	0.965	0.924	0.867	0.791	0.694	0.582	0.446	0.280	0.000
0.0	0.990	0.965	0.925	0.870	0.795	0.700	0.590	0.455	0.290	0.000
−0.1	0.990	0.966	0.927	0.873	0.800	0.706	0.598	0.465	0.300	0.000
−0.2	0.990	0.966	0.928	0.876	0.804	0.712	0.606	0.474	0.310	0.000
−0.3	0.990	0.967	0.930	0.879	0.809	0.718	0.614	0.484	0.320	0.000
−0.4	0.990	0.967	0.931	0.882	0.813	0.724	0.622	0.493	0.330	0.000
−0.5	0.990	0.968	0.933	0.885	0.818	0.730	0.630	0.503	0.340	0.000
−0.6	0.990	0.968	0.934	0.888	0.822	0.736	0.638	0.512	0.350	0.000
−0.7	0.990	0.969	0.936	0.891	0.827	0.742	0.646	0.522	0.360	0.000
−0.8	0.990	0.969	0.937	0.894	0.831	0.748	0.654	0.531	0.370	0.000
−0.9	0.990	0.970	0.939	0.897	0.836	0.754	0.662	0.541	0.380	0.000
−1.0	0.990	0.970	0.940	0.900	0.840	0.760	0.670	0.550	0.390	0.000

Column strength depends on the magnitude of bending that accompanies compression. As the column approaches failure, the compressive strains change to tensile strains in one part of the section. At failure, the latter strain attains greater extreme value than the strain at the extreme compression fiber. The resulting strain, characteristic for the strain condition at buckling, is represented by the relation $\sigma_{cr} = f(\mu)$. The effect of the tensile strains on this relation suggested to the writer a connection between the tension stress-strain and the relations $\sigma_{cr} = f(\mu)$ of various metals. If the relation between the tension stress-strain curve and the curve of a metal, $\sigma_{cr} = f(\mu)$, is known, the latter curve of another metal can be derived from its tension stress-strain diagram.

The relative magnitude of the yield stresses and the relative width of the stress ranges between the proportional limit and the yield stress influence the shapes of the stress-strain curves greatly—hence the shapes of the curve of the elasticity modulus variation. To study the relation between the elasticity modulus variation in tension and compression, a representation must be found that is suitable for the comparison of the variations.

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[10] "Die Ermittlung der Formänderungsfestigkeit von Metallen durch den Stauchversuch," by Erich Siebel and Anton Pomp, *Mitteilungen aus dem Kaiser-Wilhelm-Institut für Eisenforschung*, Vol. 9, Stahleisen, Düsseldorf, 1927, p. 164.

As a rule, the width of the stress range between the proportional limit and yield stress is different for tension and compression and varies widely for various metals. If these different stress ranges, between the proportional limit and yield stress, are represented for various metals by the same length, a suitable scale is found for the comparison of the variation of the elasticity

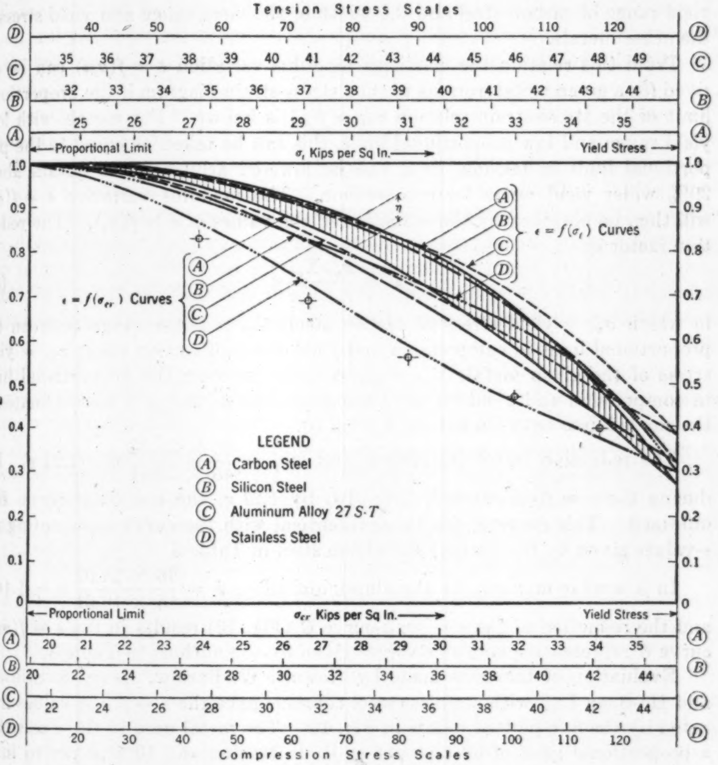

FIG. 10.—$\epsilon = f(\sigma_t)$ AND $\epsilon = f(\sigma_{cr})$ CURVES

moduli. In this representation each metal has its own stress scales along the abscissa axis of the coordinate system (see Fig. 10). The ordinates represent the ratio between the varying and constant values of the elasticity moduli. The scale of the ordinates is the same for all metals.

In this representation the differential curves $\epsilon = f(\sigma_{cr})$ of the typical tension stress-strain diagrams of the three metals investigated by the authors are nearly identical except for the vicinity of the yield stress. The curves of these metals, $\epsilon = f(\sigma_{cr})$, plotted from Table 3 are flatter, more apart, and follow the general trend of the tension modulus variation curves.

Fig. 10 indicates that the shape of the curves is independent of the constant values of the elasticity moduli and that the tension and the modulus variation curves of each metal, $\epsilon = f(\sigma_{cr})$, form a loop. The depth of these loops widens with the increase of the yield stress and the breadth of the range between the proportional limit and the yield stress. The depth of the loops is about in the proportion as the product of the ratios of yield stress and width of the yield range of carbon steel and the width of the yield range and yield stress of the other metals.

From this relation the elasticity modulus variation $\epsilon = f(\sigma_{cr})$ can be derived for a given metal from its tension stress-strain diagram if the proportional limit of the stress-strain relation $\sigma_{cr} = f(\mu)$ is known. For metals with long yield range and low proportional limit, this can be taken as equal to the proportional limit in tension, or it can be lowered arbitrarily to obtain about 20% wider yield range for compression. The modulus variation $\epsilon = f(\sigma_{cr})$ will then be obtained by the reduction of the values of $\epsilon = f(\sigma_t)$. The reduction factor is

$$\psi = \frac{\sigma_{yc}\, X_{yr}}{\sigma_{yx}\, C_{yr}}\, \eta \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (52)$$

in which σ_{yc} = yield stress of carbon steel; C_{yr} = stress range between the proportional limit in compression and yield stress of carbon steel; σ_{yx} = yield stress of the given metal; X_{yr} = stress range between the proportional limit in compression and yield stress of the given metal; and η = the ordinates of the shaded area between curves *A*, Fig. 10.

The reduction factor for silicon steel is $\psi = \frac{36 \times 25.12}{45 \times 16.61}\, \eta = 1.21\, \eta$. Reducing the $\epsilon = f(\sigma_t)$-curve *B* (Fig. 10) by $1.21\, \eta$, the $\epsilon = f(\sigma_{cr})$-curve *B* is obtained. This curve is practically identical with the curve representing the ϵ-values given by the authors for silicon steel in Table 3.

In a similar manner, for the aluminum alloy, $\psi = \frac{36 \times 24.07}{45 \times 16.61}\, \eta = 1.16\, \eta$, and the reduction of the $\epsilon = f(\sigma_t)$-curve *C* (Fig. 10) results in the $\epsilon = f(\sigma_{cr})$-curve *C* representing similar ϵ-values given by the authors in Table 3.

Evaluation of the tests made by Howard W. Barlow, Henry S. Stillwell, and Ho-Shen Lu[11] with stainless steel columns gave the $\epsilon = f(\sigma_{cr})$-values indicated by the five plotted points in Fig. 10. The metal used in these tests had a proportional limit of 30 kips per sq in. in tension and 10 kips per sq in. in compression. The yield stress was 130 kips per sq in. The writer reduced the $\epsilon = f(\sigma_t)$-curve *D* (Fig. 10) by $\psi = \frac{36 \times 120}{130 \times 16.61}\, \eta = 2.0\, \eta$ and obtained the $\epsilon = f(\sigma_{cr})$-curve *D*. The general trend of this curve agrees with the test results, and the differences are about 5%.

In Eq. 52, reduction factor ψ is based on carbon steel. This factor, of course, can be based on any other steel for which the relations $\epsilon = f(\sigma_t)$ and $\epsilon = f(\sigma_{cr})$ are known. The writer does not claim that reduction factor ψ applies rigorously in every case. More extensive studies are required to refine the expression for the reduction factor.

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[11] "Strength Investigations of Thin Stainless-Steel Sections," by Howard W. Barlow, Henry S. Stillwell, and Ho-Shen Lu; presented at the 1940 Annual Meeting of the Institute of the Aeronautical Sciences.

The significance of obtaining $\epsilon = f(\sigma_{cr})$ from the tension test is that to undertake extensive column testing programs for each metal of different proportional limit and yield strength is not practical. Tests indicating the stress-strain relation in tension, on the other hand, are simple, inexpensive, and rapid. The tension test shows more clearly and consistently the plastic behavior of metals than any other. In testing for buckling there are endless factors that influence the test results. The compression member in the structure is mostly in an entirely different stress condition than the test specimen. Imperfections in the fabrication and erection, the moments at the ends, and along the member contribute more to the uncertainties of the factor of safety than deviations in the ϵ-values. The design for buckling can be based on the reduced tension-stress-strain diagram.

The authors propose $\zeta = 0.5 (\epsilon + \sqrt{\epsilon})$ as the modulus factor for plates for all stress conditions. Although this is a practical generalization, it should be kept in mind that the modulus variation is different for homogeneous stress conditions, like uniform compression, and nonhomogeneous stress conditions, like bending. For stiffened plates, the modulus variation changes not only with the stress condition but also with the make-up of the plate. Poisson's ratio is 0.3 for the elastic part and 0.5 for the plastic part of the deformations. It appears that only tests can show the stiffness variation of plates of more complicated arrangement under the various stress conditions. Such research is available for types not used in bridge and building construction. The authors' proposal meets the requirements of structural design safely.

By deriving simple and practical rules for safe and economical design of the most complex structures, the authors have made an outstanding contribution to promote progress in structural design. The writer has attempted to suggest a way to remove obstacles to a general use of the material presented in the paper.

Joseph S. Newell,[12] **Esq.** (by letter).—The points of view of the civil and aeronautical structural designer being different, there is little that a practitioner in the latter field can criticize or generalize upon in this paper. From his point of view, some of the procedures described would lead to structures too conservatively proportioned for use in aircraft, but he realizes that rigidity of structure is of greater importance in civil engineering practice than is economy of weight, and concludes that a rationalization of the "rough-and-ready" rules for the stability of structural elements formerly, and probably currently, used by bridge designers is not only justifiable but urgently needed. As such, the paper merits the consideration of practicing structural designers, and of those members of regulatory committees whose duties entail the preparation or modification of design specifications.

Minor faults, based on differences of opinion or practice rather than errors in fundamentals, can be enumerated. The last sentence in the second paragraph of the "Synopsis," for instance, leads one to believe that once a structural element has formed visible waves, a small increase in the load acting upon it will cause its failure. The aircraft designer knows this is not so. A wide, thin plate subjected to compression, if properly stiffened by members running in the direction of the load, may be badly buckled in the region between stiffeners yet carry without failure a stress on, and adjacent to, the stiffeners of five, ten, fifteen or even twenty times that which caused the central part of the plate to buckle. Eq. 1 may be said to indicate the compressive stress at which the central part of such a plate will buckle, but it does not give the stress at which failure of plate and stiffener will necessarily occur. Perhaps it would be well to emphasize that where stiffened plates are used as structural elements, a load which would cause an average stress over plate and stiffener in excess of σ_{cr} given by Eq. 1, actually causes buckling in the central part of the plate when σ_{cr} is reached, and a redistribution of stress near the stiffeners for stresses exceeding σ_{cr}. For such panels, the critical stress intensity may be the yield point of the material, or the stress at which the stiffener and an effective width of sheet will fail as a simple column.

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[12] Prof. of Aeronautical Structural Eng., Mass. Inst. of Tech., Cambridge, Mass.

The "effective width" acting on either side of the stiffener in such a case would be found by taking one half the value of d computed from Eq. 9, for a plate of given thickness, t, and a stress in the stiffener, σ. However, since the general public is not accustomed to traveling across structures so proportioned that they wrinkle under load, the civil engineer cannot take advantage of the savings possible when members are proportioned by the effective-width procedure. He must look upon wrinkling of a plate as a "failure," and proceed accordingly. Eq. 15 enables him to do this conservatively and rationally by relating stiffener size to plate size in such a manner that both will become elastically unstable under the same stress. How much more consistent the proportioning of structural elements could be if the modern structural engineer were to take advantage of rational procedures such as this instead of abiding by the dicta of his handbooks, using ratios of $\frac{d}{t} = 30, 50, 100$, or other round numbers.

Plate girder web plates may also buckle due to shear without causing failure of the girder, if the chords and stiffeners are correctly proportioned. The waves formed in such circumstances simply indicate that the compressive principal stress induced by shear in the plate has exceeded the intensity compatible with elastic stability. However, a much greater shear can be carried by the plate, since it will resist by developing large diagonal tension stresses. These stresses will tend to pull the chords of the girder together and they may cause large bending moments in the chords, and large compressive stresses in the vertical stiffeners. Here again, wrinkling of the plate simply means a redistribution of stress in the member, and a competent designer, appreciating this, can proportion a structure to be safe whether the web is "shear-resistant," or "tension-field." It is appreciated that the equanimity of truck drivers might suffer were panels in a bridge suddenly to go from one state to the other as a heavily-loaded truck passed, but the fact that structures may be impracticable does not make them impossible and the more progressive engineers should be interested in rational methods for determining where the boundary between the practicable and impracticable actually lies.

The section on "Vertical Stiffeners of Webs of Plate Girders" provides a much more rational attack on that problem than is available to a majority of designers, and the subsequent section on "Horizontal Stiffeners of Webs of Plate Girders" provides the American engineer a means of proportioning stiffeners of a type which, because of a lack of rule-of-thumb procedures in his handbooks, he seldom employs, useful though they might be. The curves presented, and the illustrative examples embodying the use of three different materials, should prove invaluable to the structural engineer interested in the use of aluminum alloy or high-strength steel.

In closing, the writer wishes to commend the authors on their presentation of rational means for proportioning plates so that they will be elastically stable, and on their simplification of some of the more tedious equations and methods in the form of working curves. Whether the methods they describe will be adopted immediately by their professional brethren, or whether it will remain for the colleges to present the material to future students so that it may eventually be accepted, remains to be seen. In either case, the authors have taken steps in the right direction, and they should be commended for their pioneering effort as well as for the material covered.

HAROLD D. HUSSEY,[13] M. AM. SOC. C. E. (by letter).—The "Bibliography" emphasizes the fact that this subject has received very little discussion in the United States. Of the fifty-two items listed, only two were published in this country—one book and one paper. (The latter, (33), is a short, concise discussion of several practical problems and deserves study by structural engineers.) Four others were published abroad in English. The remaining forty-six items are in foreign languages.

The paper is welcomed as a contribution within easy reach of American engineers. It confirms present practice with respect to problems in buckling of plates as found in standard specifications, with modifications as noted herein.

The general expression for elastic stability of steel plates subject to compression or shear, as given by the authors in Eq. 1, can be expressed (assuming $E = 29{,}000{,}000$ and $\frac{1}{m} = 0.30$) as:

$$S_{cr} = 26{,}200{,}000\, k\, \zeta \left(\frac{t}{d}\right)^2 \qquad (53)$$

in which S_{cr} = critical compression or shear. Solving for $\frac{d}{t}$,

$$\frac{d}{t} = 5{,}120 \sqrt{\frac{k\,\zeta}{S_{cr}}} \qquad (54)$$

If S_{cr} is assumed to be equal to the actual stress S times a factor of safety, and

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[13] Designing Engr., Am. Bridge Co., New York, N. Y.

k and ζ are evaluated, the thickness ratio can be expressed in the general form

$$\frac{d}{t} = \frac{\text{coefficient}}{\sqrt{S}} \qquad (55)$$

The authors have given the values of k and ζ for plates subject to compression and shear.

Table 4 contains values of $\frac{d}{t}$ for column plates and outstanding flanges. Under the heading "Vertical Stiffeners of Webs of Plate Girders: Spacing of Vertical Stiffeners," proposed values of the coefficient in Eq. 55 are given for intermediate stiffener spacing and for girders requiring no intermediate stiffeners.

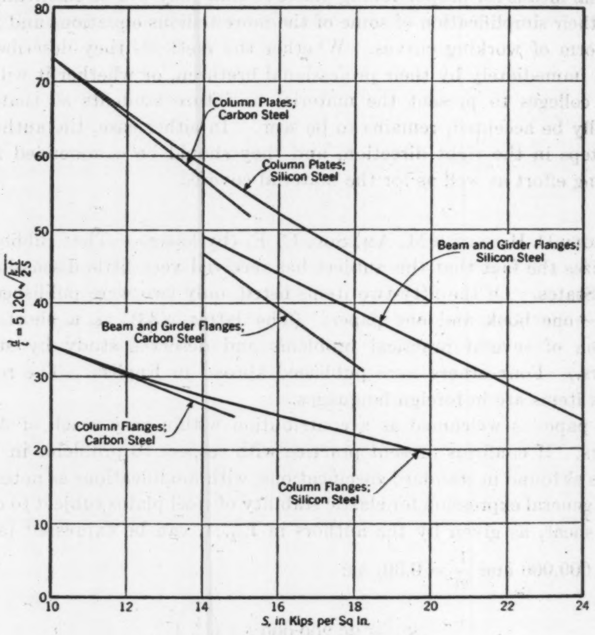

FIG. 11.—ALLOWABLE $\frac{d}{t}$ RATIOS

Values of $\frac{d}{t}$ for outstanding parts of columns, as given in Table 4(*b*), are open to question. The minimum value of k (this assumes that the outstanding part is free to rotate) was used in computing Table 4(*b*) in order to allow for initial curvature. This is equivalent to an assumption that the column has no torsional rigidity, which is not true in a practical case. It would seem more

reasonable to assume an elastically built-in edge and a value of k equal to twice the value assumed.[14] This would increase the values of $\frac{d}{t}$ given in Table (4b) by 41.4%.

The width-to-thickness ratios given in Table 4 show an important characteristic when examined in the light of current specifications. The latter fix $\frac{d}{t}$ ratios for maximum allowable unit stresses and, for smaller stresses, provide that $\frac{d}{t}$ may be increased in proportion to the square root of the ratio of allowable to actual stress.[15] The values of $\frac{d}{t}$ in Table 4, when plotted in Fig. 11, show an approximately uniform variation. The same is true of values of $\frac{d}{t}$ for depth-to-thickness ratios for girder webs, plotted in Fig. 12. This characteristic is due to the influence of the "modulus factor" ζ. A study of these curves shows that, for stresses less than the allowable, $\frac{d}{t}$ should be increased in direct proportion to the ratio of allowable to actual stress.

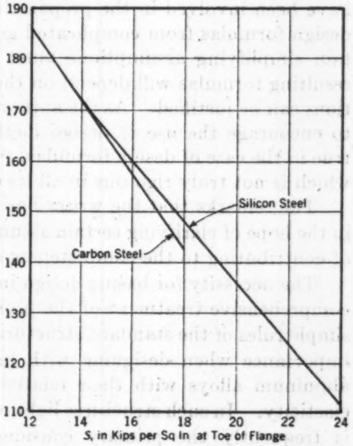

FIG. 12.—ALLOWABLE CLEAR DEPTH-TO-THICKNESS RATIOS FOR GIRDER WEBS

The authors have developed web slenderness ratios of 165 and 136 for girder webs of carbon and silicon steels, respectively (see heading "Chapter 3.—Design of Webs for Plate Girders"). These values compare with 170 and 145 used in current specifications.[16] The authors have used the clear distance between the flange angles as the assumed depth of web. A value of 24 for the buckling coefficient k, assumed by them, is for a plate simply supported on all sides (that is, free to rotate). The flange angles will provide some restraint to the free rotation of the web. In deriving the current values of web slenderness, it was assumed that this restraint extended a distance of five times the thickness of the web away from the flange.[17] This justifies the higher values in current specifications.

The authors have developed a new formula for the spacing of intermediate stiffeners of plate girders (Eq. 35a). Because of its simplicity and a greater accuracy, it is recommended as a substitute for the formulas in current use.

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[14] "Theory of Elastic Stability," by S. Timoshenko, First Edition, McGraw-Hill Book Co., Inc., New York and London, 1936, p. 342.

[15] A. R. E. A. Specification for Steel Railway Bridges, 1938, paragraphs 405 and 431.

[16] *Loc. cit.*, paragraphs 431 and 1602.

[17] "Elastic Stability of Plates Subjected to Compression and Shear," by O. E. Hovey, Hon. M. Am. Soc. C. E., *Proceedings*, A. R. E. A., Vol. 36, 1935, p. 721.

H. N. Hill,[18] Assoc. M. Am. Soc. C. E. (by letter).—The problem dealt with in this paper is one that should command the increasing attention of structural engineers. The problem consists of expressing the solutions for various instability problems connected with structural design in such a form that they may be readily understood and conveniently applied by the designing engineer. The paper is of particular value since it treats a subject which, for many purposes, is inadequately covered in current specifications for structural steel design. With the exception of the column formulas, the rules in current structural steel specifications covering features of design controlled by stability considerations have been formulated to obtain extreme simplicity, at the expense of accuracy.

The authors have attacked the problem of deriving more rational methods for the design of stiffened and unstiffened column plates and plate girder webs, by application of solutions obtained from the theory of elastic stability. The reader cannot fail to be impressed with the great amount of work that must have been involved in the preparation of such a paper. In the derivation of design formulas from complicated general solutions of stability problems, certain simplifying assumptions must necessarily be made. The value of the resulting formulas will depend on the extent to which the simplifying assumptions can be justified. As the authors state, experimental verification is needed to encourage the use of design methods thus obtained. This is particularly true in the case of design formulas, such as Eq. 31, the theoretical derivation of which is not truly rigorous in all its aspects.

The remarks that the writer has to make concerning this paper are offered in the hope of clarifying certain assumptions made by the authors and, perhaps, of contributing to the completeness of the subject.

The necessity for basing design involving stability considerations on a more comprehensive treatment of the problems involved than that embodied in the simple rules of the standard structural steel specifications becomes of increasing importance when designing with high-strength alloy steels or high-strength aluminum alloys with their relatively high ratio of strength to modulus of elasticity. In such structures light weight and consequent economy of material is frequently the primary consideration. Design in the aircraft industry probably represents the extreme case in which a high strength-to-weight ratio is the primary objective. It is in connection with this industry that the greatest advances have been made in methods of structural design involving considerations of stability. Perhaps the extensive bibliography which the authors have included might be augmented by reference to the numerous publications of the National Advisory Committee for Aeronautics on stability problems in general and stiffened flat plates in particular. Many of these publications are translations of papers that appeared originally in foreign languages.

In connection with Table 1, a "partly clamped" edge condition is assumed for the case of shear in panels having length-to-width ratios of less than 0.5. Since the length-to-width ratio β represents the proportions of a plate, a plate having a β-value less than one half can be expressed as having a β-value greater than 2 simply by transposing the length and width terms. The justification for assuming partial edge fixity for plates having β-values less than 0.5 is not obvious. This is an assumption that may require experimental verification.

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[18] Research Structural Engr., Aluminum Co. of America, New Kensington, Pa.

In Table 2, k-values are given that are applicable to plates subjected to combined bending and shear. It should be remembered, however, that these k-values are not applicable to plates subjected to other combinations of stress. Similar values are available for the case of a plate with simply supported edges subjected to combined uniform compression in one direction and shear (11). Fig. 13 shows a curve representing these values compared with a similar curve for the values given in Table 2. They indicate that the presence of the uniform compressive stress produces a greater reduction in the critical shear stress than is the case for combined bending and shear.

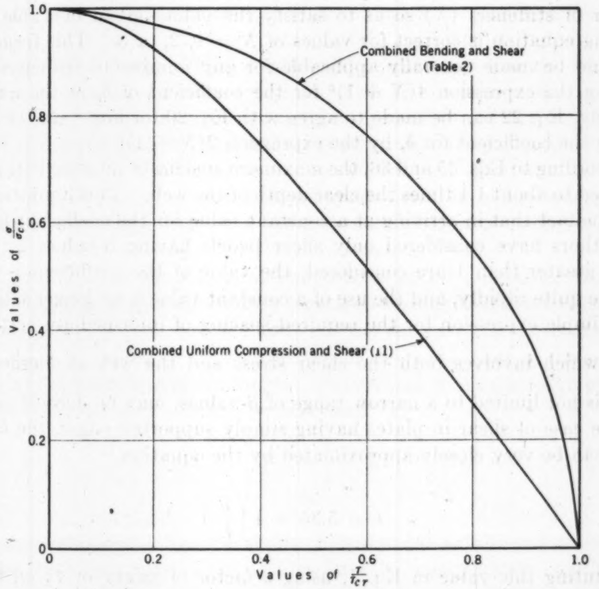

FIG. 13.—CRITICAL STRESSES FOR A SIMPLY SUPPORTED FLAT PLATE SIMULTANEOUSLY SUBJECTED TO DIRECT STRESS AND SHEAR

The authors state that for stresses beyond the yield strength of the material, the modulus factor ξ is theoretically zero. This is undoubtedly true for mild steel that has a definite yield point. The statement does not apply, however, to those materials that have no definite point of yielding. Experience with the aluminum alloy cited by the authors (27S-T) indicates that the modulus factor at the yield strength (stress corresponding to 0.2% set) is generally about 0.4.

In Table 3 the authors give stresses and corresponding modulus-factor values, from which the critical stress can be determined by a trial-and-error procedure. In problems involving buckling at stresses in the plastic range, the writer has found it convenient to express the relation between stress and reduced modulus in the form of a curve in which stress is plotted against the ratio of stress to corresponding reduced modulus value. The formula for critical stress (Eq. 1) can be rewritten by transposing E to the left-hand side. The values for the term $\frac{\sigma_{cr}}{E}$ can then be calculated from such an equation, and corresponding critical stress values can be picked directly from the curve.

Eq. 21 has been obtained by expressing k_s and $\Sigma \sin^2 (\pi C_s)$ in terms of the number of stiffeners (N) so as to satisfy the values given in Table 5. The resulting equation is correct for values of $N = 1, 2,$ or 3. This formula (Eq. 21) could be made generally applicable for any number of stiffeners by substituting the expression $4(N + 1)^3$ for the coefficient of δ_s in the numerator. Similarly, Eq. 22 can be made to agree with Eq. 20 for any value of N by replacing the coefficient for δ_s by the expression $2(N + 1)^3$.

According to Eqs. 35 and 36, the maximum spacing of intermediate stiffeners is limited to about 1.1 times the clear depth of the web. This limitation results from the fact that in arriving at a constant value for the coefficient in Eq. 34, the authors have considered only shear panels having β-values $\leqq 1$. If β-values greater than 1 are considered, the value of the coefficient is found to increase quite rapidly, and the use of a constant value is no longer satisfactory.

A simple expression for the required spacing of intermediate vertical stiffeners, which involves both the shear stress and the web slenderness $\frac{d}{t}$ and which is not limited to a narrow range of β-values, may be derived as follows: For the case of shear in plates having simply supported edges, the k-value of Eq. 1 can be very closely approximated by the equation,[19]

$$k = 5.35 + 4\left(\frac{d}{L_s}\right)^2 \qquad \text{(56)}$$

Substituting this value in Eq. 1, using a factor of safety of 1½ on the shear strength and simplifying, yields for the required spacing of stiffeners the equation,

$$L_s = \frac{0.865}{\sqrt{0.341\,\dfrac{1 - \left(\dfrac{1}{m}\right)^2}{E}\,\tau - \left(\dfrac{t}{d}\right)^2}} \qquad \text{(57)}$$

which can be reduced to the following equations for steel and aluminum: For steel,

$$L_s = \frac{865}{\sqrt{0.0107\,\tau - 1{,}000{,}000\left(\dfrac{t}{d}\right)^2}} \qquad \text{(58a)}$$

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[19] See "Theory of Elastic Stability," by S. Timoshenko, McGraw-Hill Book Co., Inc., 1936, p. 361.

for aluminum,

$$L_s = \frac{865}{\sqrt{0.0295\,\tau - 1{,}000{,}000\left(\frac{t}{d}\right)^2}} \ldots\ldots\ldots\ldots (58b)$$

The paper deals almost entirely with the design of column plates and plate girder webs. Brief mention is made, however, of the buckling behavior of compression flanges. Space limitations probably prevented the authors from dwelling longer on this particular stability problem. As the authors state (following Eqs. 11), the buckling behavior of compression flanges between lateral supports is vastly different from that of a column subjected to axial stress, because the tension part of the beam has a stabilizing influence on the buckling of the compression flange. This stabilizing influence involves the resistance of the beam to twisting. Design formulas which do not take into account the torsional stiffness of the beam should not be expected to provide an adequate treatment of the problem. After numerous efforts to devise a simple yet adequate design formula, the writer has come to the conclusion that if such a formula is to be generally applicable to all symmetrical or nearly symmetrical flanged beams with single webs, the equation derived from a theoretical treatment of the problem represents the simplest practical form.

As a basis for deriving a general design formula for allowable stresses in compression flanges of beams, it seems logical to assume that the unsupported length is subjected to pure bending and that the ends are free to rotate about an axis in the web normal to the neutral axis of the beam (that is, the ends are not restrained against lateral bending). The critical stress for a symmetrical I-beam under these conditions can be expressed[20] as

$$\sigma_{cr} = \pi\,\frac{\sqrt{E\,I_1\,J\,G}}{S\,L}\sqrt{1 + \frac{\pi^2\,E\,I_f\,h^2}{2\,G\,J\,L^2}} \ldots\ldots\ldots\ldots (59)$$

in which I_1 = moment of inertia about the principal axis in the web (in.^{4}); J = section factor for torsion (in.^{4}); L = laterally unsupported length (in.); I_f = lateral moment of inertia of one flange (in.^{4}); h = depth of beam (in.); S = section modulus about the principal axis normal to the web (in.^{3}); and G = modulus of elasticity in shear:

$$G = \frac{E}{2\left(1 + \frac{1}{m}\right)} \ldots\ldots\ldots\ldots (60)$$

Although Eq. 59 was derived for symmetrical I-beams, it is also a very close approximation for channel sections and nearly symmetrical I-sections, if the value used for I_f is that for the compression flange. In the case of symmetrical I-sections, I_f can be taken as one half I_1. Eq. (59) can be generalized so as to cover buckling beyond the elastic range, and written

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[20] "Theory of Elastic Stability," by S. Timoshenko, McGraw-Hill Book Co., Inc., 1936, p. 261.

$$\frac{\sigma_{cr}}{E_x} = \frac{\pi}{\sqrt{2\left(1+\frac{1}{m}\right)}} L^2 S \sqrt{I_1 \left[J L^2 + \pi^2 \left(1 + \frac{1}{m}\right) I_f h^2 \right]} \ldots (61)$$

in which E_x is the modulus of elasticity corresponding to σ_{cr}. By introducing a factor of safety, Eq. 61 can be reduced to a design formula, for any given material. If desired, the effect of lateral end restraint can be introduced by replacing the unsupported length L in Eq. 61 by an equivalent length $K L$, as is frequently done in dealing with columns. For unrestrained ends, $K = 1$, and for completely restrained ends, $K = 0.5$. Further refinement could also be obtained by introducing a coefficient to take account of different loading conditions. Values for such coefficients have been published for numerous loading conditions.[21]

Values for the torsion factor J, used in Eqs. 59 and 61, are given in some handbooks[22] for various sizes of I-beams and channels. A reasonably close approximation for this value may be obtained for any single web section by considering the cross section as composed of a series of rectangles. The torsion factor for the section is then the sum of the torsion factors of the various elements:

$$J = \frac{1}{3} \Sigma\, b\, t^3 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (62)$$

in which b is the length of the rectangle and t is the width.

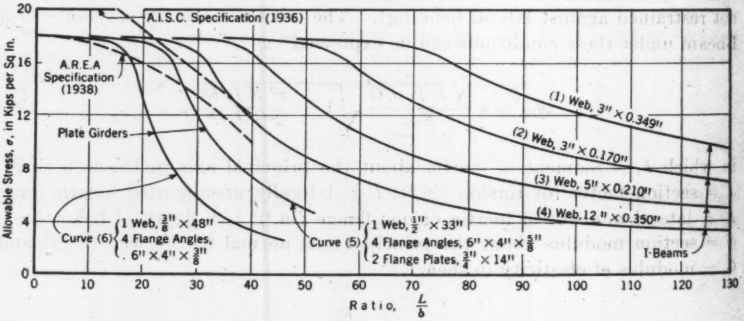

FIG. 14.—ALLOWABLE STRESSES FOR LATERALLY UNSUPPORTED FLANGES OF STEEL BEAMS WITH I-SECTION

The hopelessness of trying to obtain an adequate design formula expressing the allowable stress as a function of the ratio $\frac{L}{b}$ of the compression flange is demonstrated in Fig. 14. Curves in this figure indicate the relation between allowable stress and unsupported length for mild steel beams of various I-sections, including standard I-beams and plate girders. These curves represent values for allowable stress calculated according to Eq. 61, assuming the material to have a yield point of 36,000 lb per sq in., and an elastic limit of 32,000 lb per sq in., and based on a factor of safety of 2. The allowable stress for a compression flange, as indicated by the American Railway Engineering Association and American Institute of Steel Construction specifications, is also shown in the figure. It is evident from a comparison of the curves in Fig. 14 that the specification formulas give values for allowable stress that are ultraconservative in the case of small I-beams, and that provide a factor of safety smaller than that intended in the case of deep plate girders.

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[21] "The Lateral Instability of Deep Rectangular Beams," by C. Dumont and H. N. Hill, *Technical Note No. 601*, National Advisory Committee for Aeronautics (N. A. C. A.).

[22] Bethlehem Manual and Structural Aluminum Handbook.

The formulas given in current specifications for the allowable stress in compression flanges apply to all built-up members subjected to bending. This classification includes double web or box girders. The lateral stability of a box girder is much greater than that of a single web girder having the same flange width. This is obvious from a consideration of the greater lateral bending stiffness and torsional stiffness of the box section as compared with an I-section. It can be demonstrated[21] that, depending on the ratio of width to depth, the critical stress for a thin-wall, deep rectangular tube may be as much as three times as great as the critical stress for a solid rectangular beam of the same dimensions, loaded and supported in the same manner. In general, the lateral bending stiffness, and particularly the torsional stiffness of box sections, are relatively so great as to make a consideration of the possibility of lateral buckling of such a beam unnecessary.

F. H. FRANKLAND,[23] M. AM. SOC. C. E. (by letter).—A study of this paper can lead only to the conclusion that its contents will have a far-reaching effect in the establishment of improved and more intelligent design methods as applying particularly to plate-girder and cellular-plate constructions.

Recent investigations into the stress distribution under load of such constructions have indicated clearly that many engineers are dissatisfied with the pertinent so-called standard design methods of the past, which, to a great extent, ignored the fundamental laws of structural behavior.

This paper is a contribution of great importance to the profession in that it sets forth in a clear and understandable manner the general characteristics of buckling within the zone of elastic stability. This is of basic importance in the establishment of improved design methods whereby engineers may construct safer, more efficient, and more economical structures.

Engineers are experiencing a steadily changing relationship between the cost of labor and the cost of materials in structures, so that what may have been economic methods of design a few years ago are not necessarily economic today or tomorrow. This economic change is proceeding at an increasing tempo, and it therefore behooves the structural designer to improve his design methods by studying the results of such valuable investigations and studies as are embodied in this paper. The simplicity of the old design conceptions as applying to the elastic stability of structures, subjected to compressive stresses, no doubt relieved many designers from intellectual effort; and therefore these old conceptions are popular with some who may be expected to resist progress.

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[23] Director of Eng., Am. Inst. of Steel Constr., Inc., New York, N. Y.

The aroused interest, due to the increasing preference for plate instead of truss construction evident during the past few years, in the elastic stability of plates, and in the stress distribution in plate structures, will be aided greatly by the information presented in this paper, as will the various investigations, now current, into different phases of the subject.

LEON S. MOISSEIFF,[24] AND FREDERICK LIENHARD,[25] MEMBERS, AM. SOC. C. E. (by letter).—In the limited space available for this paper, the writers had to confine themselves to the simpler and less differentiated problems. Mr. Balog has added valuable information by bringing forth a more complex, though not rare, case.

He treats the buckling case which involves not only the simultaneous action of bending and shear but, in addition, that of axial stress. As is to be expected, the coefficient of stability k will be smaller for bending, shear, and compression stresses acting together than for bending and shear only. In Table 12 he gives the greater reduction in the values of k as the compressive stress increasingly extends over the larger part of the section. Mr. Hill likewise calls attention to the combined uniform compression and shear.

Mr. Balog also has contributed a well-reasoned presentation of a method of deriving from the elasticity modulus variation, as determined from tension stress-strain curves, the modulus variation for compression. This procedure, if verified experimentally, promises a direct and valuable approach to the problem of critical stability. The curves of the strain scales in Fig. 10, when read vertically downward, from tension to compression, leave a memorable impression of the character of the metals considered. This is especially important when one realizes that they represent by far the most important structural metals.

The writers fully agree with Mr. Hill that experimental tests are needed to verify the theoretical results derived; in the introduction to the paper they have called attention to the need of comprehensive tests. This is especially true of Eq. 31, which gives the size of transverse stiffeners of column plates already stiffened by longitudinal stiffeners. The writers have not been able to find a better method to determine the size and spacing of stiffeners in explicit form. It may be pointed out, however, that stiffeners which were designed by Eq. 31 were checked numerically by the method proposed by Mr. Seydel (29). The results were satisfactory.

Mr. Hill also calls attention to the partly clamped edge condition assumed by the writers in the case of shear in panels having a length-to-width ratio of less than 0.5. He points out that, by transposing the length and width terms, the ratio becomes 2 and thus the assumption of partly clamped edges is not obvious. The writers were particularly concerned with the design of plate girders. For this reason the length-to-width ratios were taken less than unity as is common in such structures. The width was always intended to be the distance between toes of flange angles, and the length the distance between vertical stiffeners. Shear stiffeners, when closely spaced, tend to increase the buckling stability in the compression zone. This was not taken into account in the fundamental equation, but was compensated for by assuming partly clamped edge conditions for length-to-width ratios of less than 0.5.

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The increase in k for partly clamped edges, as assumed by the writers, over that of free edges is not substantial. For a length-to-width ratio of 0.40, the increase is 2.7%; it increases to only 24% for a ratio of 0.20.

Mr. Hill also points out, with justification, that the modulus factor at the yield point should not be taken equal to zero for aluminum alloy 27S-T. It is correct, as stated by him, that for the yield strength of this alloy the modulus factor is approximately 0.4. The effect of this is, no doubt, of importance for airplane design but is of no consequence in the field of heavy structures where much higher factors of safety are used.

He also makes a valuable contribution by suggesting expressions that would make Eqs. 21 and 22 applicable to any number of stiffeners and not only to three, as limited by the writers.

He suggests different formulas for spacing of stiffeners—Eqs. 57 and 58. These equations are generally more applicable for a greater range of length-to-width ratios. The writers wish to point out, however, that their formulas are considerably simpler and suitable for practical design of plate girders. For plates, where the stiffeners can be placed so that the length-to-width ratio is considerably greater than unity, the formulas of the writers do not apply. They suggest that the equations derived by Mr. Hill be used.

Mr. Hill further discusses the twisting stability of beams. The writers are well aware of this phenomenon and its great importance in structural design. Space limitations, however, precluded a discussion of the problem. The buckling stability of compression flanges, including torsional rigidity, has been treated elsewhere.[26]

Mr. Hussey points out that, for values of $\frac{d}{t}$ for outstanding parts of columns, the minimum value of k was used in computing Table 4(b) in order to allow for initial curvature. He is of the opinion that it would be more reasonable to assume a value of k equal to twice the value assumed, which would result in increasing the values of $\frac{d}{t}$ by 41%. The writers believe that this would be too optimistic an assumption and prefer to adhere to the original values until careful tests justify an increase in the width-to-thickness ratio.

Mr. Hussey also calls attention to the fact that, for stresses less than the allowable, $\frac{d}{t}$ should be increased in direct proportion to the ratio of allowable to actual stress. This has been done in the paper.

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[26] "Design Specifications for Bridges and Structures for Aluminum 27S-T," by Leon S. Moisseiff, pp. 2 and 3, and 59 to 66.

As bridge and structural engineers, the writers had in mind the rational and economical design of structures intended to carry substantial loads and which themselves weigh many tons. They did not take into consideration the design of airplanes and similar structures where the dead weight is of the utmost importance and for which the limits of safety must be stretched to the utmost. Professor Newell correctly has called attention to the neglect of the writers to point out the residual strength left in many members after buckling has become evident. In the mind of the bridge engineer, structures in which elements have wrinkled into visible waves have lost their practical usefulness, as Professor Newell has well stated. Preoccupation with their field made the writers forget to limit properly the last sentence in the "Synopsis."

Mr. Frankland's comment is of special value because it comes from one so thoroughly familiar with the ever-active development of steel structures and the increasing demand for their improvement from the viewpoints of the designer as well as the fabricator.

The aim of the writers in presenting the paper was to call the attention of designers of engineering structures to the fact that the rules for the stability of the component parts of girders and columns, as given in most specifications, are crude and simple, and that "the very simplicity of the rule confines the application to narrow limits and hampers the design and its economy." They attempted to gather from the vast literature on elasticity, which has become available since about 1910, the most essential results relating to the stability behavior of metals, and to deduce from them formulas which would give the designer ample freedom to utilize the material in the most economical manner and which, at the same time, could be applied readily.

The writers feel obliged to the participants in the discussion for the valuable criticisms and contributions made to the paper, thereby adding to its value. The interest that this rather involved paper has evoked in engineers engaged in the design of metallic structures may serve as a sign of its timeliness. It is gratifying to note that comprehensive tests on elastic stability problems are at present being considered by interested professional bodies.

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MASONRY DAMS
A SYMPOSIUM

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FOREWORD

BY THE AUTHORS

Engineering progress made in connection with the design and construction of dams of unprecedented height—such as Boulder Dam on the Colorado River near Las Vegas, Nev., Grand Coulee Dam on the Columbia River in central Washington, and Shasta Dam on the Sacramento River in north-central California—has thrown much light on what have heretofore been somewhat uncertain problems in the science of dam building. Extensive researches, conducted in the field as well as in the office and laboratory, have developed important information regarding the exploration of foundation conditions, ways of predicting foundation deformations, methods of analyzing structural action, means of determining internal stress conditions, fundamental laws regarding heat flow, and its control in mass-concrete structures, and other basic criteria involved in the more technical phases of dam design. Similar investigations, conducted in both laboratory and field, have developed pertinent data regarding the best methods of preparing foundation formations, selecting and proportioning concrete ingredients, placing and curing mass concrete, controlling internal temperature changes, and conducting other operations of a construction nature. In view of these developments it seemed desirable to offer a carefully planned symposium on masonry dams for general discussion.

BASIC DESIGN ASSUMPTIONS

By Ivan E. Houk,[1] and Kenneth B. Keener,[2] Members, Am. Soc. C. E.

Synopsis

Basic assumptions and related technical considerations involved in the design of high and important masonry dams of the single-arch, curved gravity, and straight gravity types, built on rock foundations, are presented in this paper. Some of the statements are applicable to the design of other types of masonry dams. However, this paper does not attempt to cover, comprehensively, the fundamental criteria involved in the design of multiple-arch dams, reinforced-concrete slab and buttress dams, roundhead buttress dams, or other special types of masonry dams treated in more detail in other papers of the Symposium.

Furthermore, this paper is confined, primarily, to the basic assumptions which have either undergone appreciable modifications during recent years or have been developed as entirely new criteria for the design of important masonry dams of the aforementioned types. Basic information, such as normal stream-flow conditions at the site, magnitudes of ice pressure, maximum range of seasonal concrete temperature changes, physical properties of concrete materials, and other data which may be intelligently ascertained or predicted from readily available records, or may be determined by routine laboratory measurements, are not discussed. Assumptions involved in determining maximum anticipated flood intensities for use in designing spillway features constitute a special problem and consequently are not included.

Basic assumptions for the design of masonry dams are treated from the viewpoints of the dam site, the dam, load conditions, structural action, stability factors, and stress conditions. Details of procedures involved in dam design are not included. However, a bibliography at the end of the paper lists the more important recent articles which should be consulted by any one interested in an exhaustive consideration of the subject.

The Dam Site

The adequacy of the site constitutes the first fundamental criterion for the design of a masonry dam. A gravity structure is only as stable as the base on which it rests. An arched structure is only as stable as the walls against which

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the arch elements abut. The stability of the rock formations, which constitute the site and which, therefore, must support the dam and the loads transmitted by the dam, depends on the physical properties of the rock, the uniformity of the rock structure, the arrangement of the geological formations, the presence of sheared or faulted zones, the presence of seams or other bodies of disintegrated, unsatisfactory material, and the extent to which inadequate characteristics of the geological structure may be corrected by feasible foundation improvements.

Surface indications of bedrock conditions are often misleading. At many dam sites natural forces have materially altered the foundation formations to appreciable depths below the exposed surfaces. Consequently, the adequacy of the site must be determined by thorough explorations into the geological strata to such depths as may be affected by the construction of the dam and the subsequent filling of the reservoir. Cores of comparatively small diameter, obtained by drilling operations, may be sufficient in the case of unusually uniform and satisfactory rock formations. However, such information has sometimes been misleading or has had a tendency either to discredit the foundation or to confuse the interpretation of the data. As a general rule, the larger the diameter of the cores extracted from the rock, the more reliable is the information obtained and the greater the significance that can be attributed to the specimens. The most satisfactory methods of foundation explorations include the excavation of tunnels into the canyon walls, the sinking of shafts into the foundation strata, and the removal of large-diameter cores (all three of which permit visual inspection of the rock in place), in addition to the drilling and removal of numerous cores of relatively small diameters which do not permit visual inspection of the geological formations (1).[3]

From the viewpoint of conditions at the dam site the basic assumptions involved in the design of important masonry dams may be briefly listed as follows:

1. The rock which constitutes the foundation and abutments at the site is strong enough to carry the forces imposed by the dam with stresses well below the elastic limit at all places along the contact planes;
2. The bearing power of the geologic structure along the foundation and abutments is great enough to carry the total loads imposed by the dam without rock movements of detrimental magnitude;
3. The rock formations are homogeneous and uniformly elastic in all directions, so that their deformations may be predicted satisfactorily by calculations based on the theory of elasticity, by laboratory measurements on models constructed of elastic materials, or by combinations of both methods; and
4. The flow of the foundation rock under the sustained loads which result from the construction of the dam and the filling of the reservoir may be adequately allowed for by using a somewhat lower modulus of elasticity than would otherwise be adopted for use in the technical analyses.

Although it is of rather an elemental nature, assumption 1 is receiving more and more attention as dams are continually being planned for higher heads and higher working stresses. In the early days of dam construction, when dam cross sections were planned for maximum working stresses of about 10 or 12 tons per sq ft, the adequacy of the foundation rock from the stress viewpoint was often determined arbitrarily by geological inspection and engineering judgment. Today, when dam cross sections are being planned for maximum working stresses of 40 or 50 tons per sq ft, samples of the actual rock are taken from the foundations and abutments and tested in the laboratory, where the strength can be measured accurately. Consequently, this criterion, which has been listed as an assumption, and which formerly was often in reality merely an assumption, is now only an assumption in so far as the question arises as to whether or not the test samples accurately determine the true strength of the rock formations on which the dam is to be constructed. Present engineering practice in designing and constructing important masonry dams requires that an adequate number of representative samples of the foundation and abutment rock be taken and properly tested in the laboratory, so that the matter of adequate rock strength may be classified as measurable fundamental data rather than as a basic assumption.

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[3] Numerals in parentheses, thus: (1), refer to corresponding items in the Appendix.

Assumption 2 is true if assumption 1 is true and if the rock formations are continuous, or can be made satisfactorily continuous by grouting operations or by other methods of foundation preparation, such as washing out seams of soft material and refilling with grout, excavating unsatisfactory rock and replacing with concrete, and other measures of a precautionary nature.

Assumption 3 is somewhat open to question. It is doubtful if the rock formations at any dam site are ever sufficiently free from cracks, seams, fissures, joints, and bedding planes to be considered uniformly homogeneous. It is also doubtful if such formations are ever uniformly elastic in all directions, so that their movements under loads transmitted by the dam can be accurately calculated by formulas based on the theory of elasticity—formulas which, at best, can be considered only approximate. Furthermore, it is doubtful if methods of foundation preparation, such as the aforementioned, can ever produce an ideally homogeneous and uniformly elastic rock structure, even if they are entirely adequate to permit the construction of a safe masonry dam. However, for the present, elastic formulas, together with measurements on elastic models, constitute the only available methods of estimating the magnitude of foundation and abutment deformations. These movements, although small, may be relatively large proportions of the total dam deflections. Consequently, they constitute quantities that should be considered in the technical analyses.

Assumption 4 is probably within reasonable limits of accuracy in all cases where assumptions 1 and 2 are fulfilled. In connection with the consideration of foundation and abutment movements, it is interesting to note that measurements at the 313-ft Ariel Dam, a constant-angle arch dam on the north fork of Lewis River, southern Washington, showed rock movements at the south abutment about twice as great as those calculated in the trial load analyses (2). This difference between actual and anticipated abutment movements at Ariel Dam may have been partly due to flow, inaccuracies in determining the modulus of elasticity, or approximations in the elastic formulas. However, it is possible that the difference may have been due entirely to slippage of the geological formations along seams, cracks, fissures, or other planes of weakness not sufficiently strengthened in the abutment preparations.

THE DAM

The masonry dam, as constructed by the most approved methods of today (1940), is a safer, more reliable, and better understood structure than it was twenty, or even ten, years ago. Modern developments in manufacturing special cements, practical refinements in controlling the quality, proportioning, and mixing of concrete ingredients, and continued progress in improving mechanical equipment and construction procedures have eliminated many heretofore sources of uncertainty. These advances have gone hand in hand with the evolution of design methods. They are reflected in both purchase and construction specifications. Their effects are evidenced by betterments in the quality and economy of construction which characterize the more recent structures. Perhaps the most noteworthy aspects of these accomplishments are the closer approach to uniformity of the concrete in place, so that now the designer may consider concrete, more than ever before, as a satisfactory and dependable material for use in constructing important masonry dams.

The inherent limitations of concrete in respect to volume constancy have been largely offset by the introduction and use of special cements of low-heat, modified, or puzzolan-portland types (3), by building massive structures in smaller blocks or units, by artificially cooling the concrete to stable thermal conditions when necessary, and by grouting the separate units together to form a finished monolithic structure (4). The location and spacing of the contraction joints that form the boundaries of the units are determined by positions of foundation irregularities; locations of penstocks, outlets, and sluiceways; and the maximum size of individual blocks within which cracks from volume changes may not be expected to occur. The joints are filled with cement grout, injected under pressure. Cores taken from completed dams, as well as extensive laboratory investigations, have shown that the grout actually penetrates to all parts of the joint areas. In some cases the penetration was so satisfactory that joint locations could scarcely be identified in cores taken across the joints. Escape of liquid grout and leakage of water from the joints are prevented by flexible strips of non-corrodible sheet metal embedded in the concrete on both sides of the joints.

The use of economical lean mixes of low slump, containing large-size aggregate, has made uniformity of all ingredients and close control of concrete manufacture imperative. More care is continually being exercised in designing mixtures, selecting and processing aggregates, conducting mixing operations, conveying the product, and placing the concrete in the dam. More rigid control of sand gradation is being obtained by fractionation in classifiers and recombination, as at Grand Coulee Dam, in Washington, or by the addition of fine blending sand. At Marshall Ford Dam, in Texas, the equivalent of a blending sand was produced by reduction of excess coarse sand in rod mills. Segregation and breakage of coarse aggregate are avoided by more careful methods of handling and storing. Better stabilization of moisture content is obtained by providing ample drainage facilities at the aggregate stock piles, particularly in the case of sand storage. More satisfactory proportioning and mixing of ingredients are obtained by using improved weighing batchers, water meters, automatic batch recorders, sequence charging of mixers, and mixer blade designs of more appropriate shape and arrangement. The poor adaptability of the "torque-type" consistency meter for registering changes in consistency of stiff mass concrete has been overcome at Grand Coulee Dam by the development of a meter that depends for its operation on the relation between the consistency of the batch and its tendency to tilt the mixer on the trunnions.

Improvements in conveying concrete have included the substitution of buckets, pumps, and other preferred methods of transportation for belt conveyers and chuting systems. More satisfactory compaction in placing is being secured by internal vibration whenever possible. Construction joint surfaces are being prepared more carefully before adding new concrete, so as to insure higher bonding strength along contact planes.

Considering the most approved methods of constructing masonry dams, as well as the aforementioned developments in methods of manufacturing, transporting, and placing concrete, the principal basic assumptions involved in the design of important masonry dams, from the viewpoint of the physical properties of the dam, may be listed briefly as follows:

5. The base of the dam is thoroughly keyed into the rock formations along the foundation and abutments;

6. Construction operations are conducted so as to secure satisfactory bond between the concrete and rock materials at all areas of contact along the foundation and abutments;

7. The concrete in the dam is homogeneous in all parts of the structure;

8. The concrete is uniformly elastic in all parts of the structure, so that deformations due to applied loads may be calculated by formulas derived on the basis of the theory of elasticity, or may be estimated from laboratory measurements on models constructed of elastic materials;

9. Effects of flow of concrete may be adequately allowed for by using a somewhat lower modulus of elasticity under sustained loads than would otherwise be adopted for use in the technical analyses;

10. Construction joints are properly grouted under adequate pressures, or open slots are properly filled with concrete, so that the dam may be considered to act as a monolith; and

11. Sufficient drains are installed in the dam to reduce such uplift pressures as may develop along areas of contact between the concrete and rock materials.

Assumptions 5 to 11, inclusive, need little comment. Assumption 5, regarding the keying of the dam into the rock formations, should always be feasible. Assumption 6, regarding the securing of satisfactory bond along areas of contact between the masonry and the rock, may be difficult of attainment at sites where the geological strata consist of shale or other not wholly satisfactory formations. In such cases sufficient stability may be assured by keying the dam into the rock to more than usual depths. Assumptions 7, 8, and 9 are believed to be within reasonable limits of accuracy in the case of the most approved methods of construction used in the United States. They may be somewhat uncertain in the case of some foreign masonry dams where different mixtures of concrete are used in different parts of the dam. These assumptions were usually more or less open to question during the early history of masonry dam construction. However, modern researches in methods of proportioning ingredients, mixing, conveying, and placing mass concrete have eliminated many of the uncertainties regarding the use of concrete as a structural material. Assumptions 10 and 11, regarding the grouting of construction joints, filling of open slots, and installation of foundation drains, are matters of design and construction which should always be obtainable.

LOAD CONDITIONS

Load conditions at masonry dams may be classified as "normal" and "abnormal." Normal dead loads include concrete weights, weights of gates, bridges, or other superimposed features, and weights of such materials as may be deposited on the sloping faces of the dam. Normal live loads include reservoir-water pressures, tailwater pressures, uplift forces, and forces caused by seasonal changes in concrete temperature. Abnormal loads, the most important of which are all of a live-load nature, may include ice pressures, increases in fluid pressures during silt flows, earthquake shocks, construction loads, and any other applied loads of unusual or temporary nature. Construction loads include forces produced during grouting operations, internal stresses caused by artificial cooling operations, and internal stresses caused by the generation of chemical heat during the curing of the concrete. In locations where severe climatic conditions occur during the winter months, and where relatively high reservoir surfaces are maintained at such times, ice pressures may be considered as normal live loads rather than as abnormal loads.

Water pressures may be calculated accurately for given water-surface levels. Unit concrete weights may be determined accurately from laboratory samples. Weights of features superimposed on the dam, weights of materials deposited on the slopes of the dam, seasonal concrete temperature changes, and maximum ice pressures may be satisfactorily predicted from available records. Consequently these load conditions, the first two of which usually constitute the most important normal loads at masonry dams of massive arch, arched gravity, and gravity types, need not be discussed herein. In the case of high storage dams such ice loads as may develop during the winter months usually occur at elevations considerably lower than the top of the dam so that the ice pressures, together with the water pressures caused by the accompanying partial-depth reservoir loads, are usually less severe loading conditions than those existing when the reservoir is filled to the top of the dam.

Some questions have arisen in the past regarding increases in fluid pressures which may occur due to the presence of large proportions of silt, sand, or gravel in flood flows reaching the dam. The Bullards Bar Dam, a power and debris-control dam built on the North Fork of the Yuba River in California, in 1924, was designed for a fluid pressure of 90 lb per cu ft (5). Measurements at the Miami Conservancy District flood-control dams of southwestern Ohio (6), and also at the Tieton irrigation supply dam of central Washington (7), showed that horizontal pressures in the sand and gravel portions of the embankments did not exceed hydrostatic pressures, that silt materials in the cores consolidated within comparatively short times after deposition, and that horizontal pressures in the silt deposits decreased to hydrostatic values during the periods of consolidation.

In the case of a relatively high storage dam, the greater portion of the debris load is deposited at the upper end of the reservoir so that only comparatively small quantities of silt reach the dam where they may produce increases in horizontal pressure. These increases in horizontal pressure occur only during the passage of the flood and during the period of consolidation, which begins as soon as the flood subsides. Consequently, unless floods occur in close succession, the depths at which increased fluid pressures may be exerted against the upstream face of the dam are relatively small proportions of the total height of the structure. Analyses for several high masonry storage dams have shown that effects of increased horizontal pressures caused by silt conditions are negligible in comparison with effects of earthquake shocks. In designing relatively low diversion dams on streams carrying large proportions of silt, it may sometimes be desirable to consider fluid pressures greater than those caused by clean water.

Recent investigations of load conditions at masonry dams have considered uplift forces along areas of contact between the dam and the rock formations, uplift pressures at construction joints and within the pores of the concrete, forces due to abnormal changes in concrete temperature, forces produced during grouting operations, and abnormal loads caused by the occurrence of severe earthquake shocks.

Measurements at existing masonry dams have shown that uplift forces develop at the base of the dam, even if the foundation grouting operations have been unusually thorough and extensive and although numerous foundation drains have been provided beneath the upstream portion of the dam. However, measurements at Norris, Owyhee, and Boulder dams have shown that such uplift forces may be materially reduced by the installation of additional drains at the areas where the uplift pressures develop. Consequently, in planning masonry dams for sites where satisfactory rock foundations exist, and where approved grouting and draining provisions are specified, it is not necessary to design the cross sections for full uplift pressures acting over the entire area of the base. Uplift investigations conducted during the six years 1933–1939 have shown no necessity for changing the conclusion previously expressed—namely, that it seldom is necessary to apply a straight-line pressure distribution from reservoir head to tailwater head to more than two thirds the area of the base (8) (9). Of course, unusually severe uplift conditions must be assumed in designing masonry dams to be built on porous sandstone foundations, shale beds, or other not wholly satisfactory geological formations. Thus far, measurements at existing masonry dams have shown no serious uplift pressures at construction joints or within the pores of the concrete.

Internal stresses caused by artificial cooling operations may be kept within safe limits by building the dam in blocks or columns, separated by contraction joints as previously mentioned, and grouting the joints after the cooling has been completed (10). Stresses caused by increases in temperature after grouting are compression. Such stresses are beneficial since they tend to tighten the joints and produce a more monolithic structure. In the case of arch dams they also tend to produce upstream movements that work against the reservoir load. Forces produced by grout pressures during joint grouting operations must be carefully considered in planning the grouting procedure. If the grout is injected under too high pressures, the concrete columns may be cracked by shearing or bending stresses. Furthermore, in the case of arch dams the cantilever sections may be deflected upstream until horizontal cracking occurs along the downstream face. Formulas for calculating the flow of heat in masonry dams were developed during the design of Boulder Dam (11). Studies of seasonal and curing temperature changes in actual dams were published in 1930 and 1931 (12) (13).

The major development in American masonry dam design during the ten years 1929–1939, from the viewpoint of load conditions, has been the consideration of effects produced by severe earthquake shocks. Earthquake effects have been included in designing practically all large masonry dams in Japan since the Kwanto earthquake of September 1, 1923. However, the first important masonry dam in America in which earthquake loads were carefully considered in designing the cross section was the 328-ft Morris Dam, completed by the City of Pasadena, Calif., in 1934 (14). Formulas for analyzing effects of earthquake accelerations on load conditions at masonry dams were developed in connection with the work of the Bureau of Reclamation on the design of the 220-ft Madden Dam on the Chagres River, Panama Canal Zone, completed in 1934, and the 726-ft Boulder Dam, on the Colorado River near Las Vegas, Nev., completed in 1936 (15).

Accelerations caused by earthquakes may affect the stability and internal stress conditions of masonry dams, even if the epicenters are some distances from the dams. When the analyses of earthquake loads were first begun, only horizontal accelerations were considered. More recently effects of vertical as well as horizontal accelerations have been included in the stress and stability analyses. Effects of vertical accelerations are especially important in designing gravity dams where no arch action is available to assist in carrying the increased loads. In connection with the consideration of vertical earthquake vibrations at gravity dams, it should be noted that the maximum downward acceleration, as well as the maximum upward acceleration, must be analyzed. The maximum effect on sliding factors during the full condition of the reservoir is produced when the acceleration is acting downward, whereas the maximum effect on stresses is produced when the acceleration is acting upward. In the case of horizontal accelerations, maximum effects on both sliding factors and stresses during the full condition of the reservoir are produced when the acceleration is acting in an upstream direction.

Earthquake accelerations cause changes in sliding factors because uplift pressures are not assumed to be changed by the earthquake forces.

Considerations of earthquake effects have usually been based on assumptions of maximum accelerations equal to one tenth of gravity, periods of vibration of 1 sec, and horizontal vibrations acting normal to the longitudinal axis of the dam. These assumptions are usually considered to be severe enough without including allowances for such resonance as may occur. Furthermore, some increases in stress conditions and some encroachments on the factors of safety of masonry dams are usually considered permissible for such temporary and abnormal load conditions, so that actual increases in cross sections required by the consideration of earthquake accelerations are seldom of appreciable magnitude. Available records of earthquake occurrence are not sufficient to justify the conclusion that any section of the United States may not experience such phenomena sometime in the future.

From the viewpoint of load conditions, some of the basic assumptions involved in the design of important masonry dams may be listed briefly as follows:

12. Effects of increases in horizontal pressures caused by silt contents of flood waters usually may be ignored in designing high storage dams, but may require consideration in designing relatively low diversion structures;

13. Uplift forces adequate for analyzing conditions at the base of the dam are adequate for analyzing conditions at horizontal concrete cross sections above the base;

14. Internal stresses caused by natural shrinkage and by artificial cooling operations may be adequately controlled by proper spacing of contraction joints;

15. Internal stresses caused by increases in concrete temperature after grouting are beneficial;

16. Maximum pressures used in contraction-joint grouting operations should be limited to such values as may be shown to be safe by appropriate stress analyses;

17. No section of the United States may be assumed to be entirely free from the occurrence of earthquake shocks;

18. Assumptions of maximum earthquake accelerations equal to one tenth of gravity are adequate for the design of important masonry dams without including additional allowances for resonance effects;

19. Vertical as well as horizontal accelerations should be considered, especially in designing gravity dams; and

20. During the occurrence of temporary abnormal loads, such as those produced by earthquake shocks, some increases in stress magnitudes and some encroachments on usual factors of safety are permissible.

STRUCTURAL ACTION

The structural action of a masonry dam in transmitting dead and live loads to the foundation and abutments is much better understood and much more carefully analyzed today (1940) than it was a few years ago. Recent researches, model tests, and experimental measurements at existing dams have developed much pertinent information regarding different methods of load transference and proper procedures to follow in analyzing such load transference (16) (17) (18). In this connection special acknowledgments should be made to the excellent investigational work of the Engineering Foundation Arch Dam Committee (19) (20) (21).

Engineers familiar with the latest developments in masonry dam design may now determine, in a fairly accurate manner, the distribution of load between arch and cantilever elements in thin arch dams, thick arch dams, and massive curved gravity dams. They may also analyze effects of twist and beam action in both straight and curved gravity dams, as well as effects of twist and tangential shear in both arch and arched gravity dams. It is no longer necessary to design an arch dam on the assumption that all live loads are carried by arch action, or to design a curved gravity dam on the assumption that all live loads are carried by gravity action. Furthermore, it is no longer necessary to neglect effects of foundation and abutment deformations in analyzing the structural action of masonry dams. Fairly accurate estimates of the effects of such deformations may now be included in the mathematical calculations for both arch and gravity structures.

The distribution of load in a masonry dam is determined by trial load methods. The fundamental assumption involved in the trial load method is that the load is distributed between the different systems of structural action in such a way that the resulting deflections of the different systems are equal at all conjugate points in the structure. Since the distribution which fulfils this criterion can be determined feasibly only by assuming different distributions and calculating resulting deflections until a satisfactory agreement is reached, the procedure is logically termed the "trial load" method. In an arch dam the different systems of load transfer are the arches and cantilevers. Effects of tangential shear and twist are usually analyzed by applying internal, self-balancing loads and couples to the two systems and bringing deformations into agreement in tangential and angular directions as well as in radial directions. In a gravity dam the different systems of load transfer are the cantilever elements, horizontal beam elements, and the twisted structure.

The development of the trial load method of analyzing masonry dams was begun in the Denver, Colo., office of the Bureau of Reclamation in 1923, about the time a similar procedure was being investigated in Europe. The Bureau's first use of the method in analyzing arch dams (a method which did not include considerations of rock movements, tangential shear, or twist effects) was described in a paper published by the Society in 1929 (22). A complete description of the amplified method now used by the Bureau was published in bulletin form in 1938, as one of the final reports on the Boulder Canyon Project (23).

Trial load analyses of about twenty arch dams have shown that structural effects, not previously considered in masonry dam design, exert appreciable influences on load distribution and stress magnitude. Considerations of rock movements naturally tend to increase the calculated deflections of the structure. Such considerations usually result in somewhat lower arch and cantilever stresses along foundation and abutment locations without material stress changes in the central and upper portions of the structure. The general result of including tangential shear effects in trial load analyses is a decrease in radial deflections near the crown section, an increase in radial deflections near the abutments, and a slight increase in arch stresses at the abutments without appreciable stress changes at the crown. Such effects are not important at narrow canyon sites, as at Shoshone Dam, in Wyoming, but are important at relatively wide sites, as at Gibson Dam, in Montana.

The general effect of considering twist action is a decrease in radial deflections at practically all locations, a decrease in maximum arch stress, a decrease in cantilever stress at the downstream face of the dam, and an increase in cantilever stress at the upstream face of the dam (24).

Twist and beam effects in gravity dams occur primarily along the sloping abutments, the effects usually increasing in importance as the steepness of the canyon wall increases. Such effects usually cause decreases in inclined cantilever stresses at the downstream face of the dam, increases in inclined cantilever stresses at the upstream face of the dam, horizontal compression stresses along the downstream face at the abutments, and horizontal tension stresses at the upstream face at the abutments. If not relieved by suitable construction procedure, such as the provision of vertical slots as at Grand Coulee Dam, twisting action and beam action may cause diagonal cracking at the downstream face in the higher central portion of the dam near the abutments. A comprehensive description of the use of the trial load method in analyzing effects of twist and beam action in gravity dams, including formulas, was published in England in 1937 (25).

From the viewpoint of structural action, the basic assumptions involved in the design of important masonry dams may be listed briefly as follows:

21. Effects of foundation and abutment deformations should be included in the technical analyses;

22. In monolithic straight gravity dams some proportions of the loads may be carried by twist action and beam action at locations along the sloping abutments, as well as by the more usually considered gravity action;

23. Detrimental effects of twist and beam action in straight gravity dams, such as cracking caused by the development of tension stresses, may be prevented by suitable construction procedure;

24. In monolithic curved gravity and arch dams some proportions of the loads may be carried by tangential shear and twist effects, as well as by the more usually considered arch and cantilever actions; and

25. The distribution of loads in masonry dams may be determined by bringing the calculated deflections of the different systems of load transference into agreement at all conjugate points in the structure.

STABILITY FACTORS

Until a few years ago the stability of a straight gravity dam against failure by sliding was usually determined by calculating the ratio of the total horizontal force to the total vertical force, a ratio known as the "sliding factor," and comparing this ratio with the friction coefficient for concrete sliding on concrete or concrete sliding on rock. If the calculated value of the sliding factor was less than the appropriate value of the friction coefficient at all elevations analyzed, it was assumed that the section would be stable and that the additional security provided by keying the concrete base into the rock foundation, together with the bond secured at the contact areas, would furnish adequate factors of safety at all locations. It was recognized that the tendency of the structure to move downstream under the applied reservoir load would be resisted by the shearing strengths of the concrete and rock materials in all cases where dams were built by approved methods on satisfactory foundation strata; but attempts to evaluate actual factors of safety, including restraining effects of shearing strength, usually were not made. In 1933 the late D. C. Henny, M. Am. Soc. C. E., consulting engineer on many of the dams built by the Bureau of Reclamation, proposed a method for calculating the factor of safety against downstream movement, including allowances for shearing strength (26). Since that time engineers of the Bureau of Reclamation have been considering the stability of straight gravity dams on the basis of the shear-friction factor of safety, a factor which is calculated in essentially the same manner as Mr. Henny's shear safety factor, *Q*. F. D. Scheidenhelm, M. Am. Soc. C. E., made detailed studies of shear resistance in preparing a report on the stability of the State Line Dam on Cheat River, West Virginia, in 1921, but did not publish the results of his investigations.

The basic assumptions involved in calculating shear-friction factors of safety and in using such factors to determine the stability of gravity dams against movement along horizontal planes have been set forth herein. Recapitulating briefly, the principal pertinent assumptions are numbers 3, 5, 6, and 7, regarding the homogeneity of the rock, the keying of the concrete base into the foundation, the securing of satisfactory bond, and the uniformity of the concrete.

The principal uncertainty involved in evaluating the shear-friction factor of safety is the determination of the average shearing strength of the material. The engineer must remember that the proper value to use in the formula is the shearing strength of the rock if the rock is weaker than the concrete, or the shearing strength of the concrete if the concrete is weaker than the rock. Values of shearing strength used in calculating shear-friction factors of safety for dams built by the Bureau of Reclamation have varied from about 300 to 700 lb per sq in., depending upon the characteristics of concrete and rock specimens as determined by laboratory tests. The value of the friction coefficient has been taken as 0.65 in all cases, instead of 0.70, as used in Table 9 of Mr. Henny's paper.

The aim of the Bureau of Reclamation has been to keep the minimum shear-friction factor of safety greater than 5 during the most severe condition of reservoir load combined with maximum horizontal and vertical earthquake accelerations. This is easily done in designing gravity dams of ordinary height but requires unusually careful planning in designing dams of unprecedented height, such as Shasta (in California) and Grand Coulee.

Factors of safety against failure by shear along vertical planes may be calculated by methods similar to those used in computing shear-friction factors of safety. In this case the computations for the factor of safety do not include weight effect. They depend only on total shearing forces and total shearing strengths along vertical planes.

STRESS CONDITIONS

One of the most important developments in masonry dam design during the fifteen years 1924–1939, if not the most important, has been the gradual improvement in methods of analyzing stress conditions, with the accompanying gradual increase in allowable working stresses. This development in technical design procedure has been a natural result of the necessity for building higher and higher structures. Dams such as Boulder, Grand Coulee, and Shasta could not be built if working stresses had to be limited to 10 or 12 tons per sq ft, as they were in designing dams built during the first few years of the twentieth century. In dams of unprecedented height, such as those just mentioned, stresses due to dead loads alone greatly exceed the earlier assumptions of maximum allowable pressures.

Engineers engaged on the design of important masonry dams today recognize that the safety of the structure from the viewpoint of stress conditions should be judged on the basis of the three-dimensional state of stress rather than on the basis of the maximum direct stress in one direction, the condition which usually constituted the governing criterion in early examples of masonry dam design. For instance, at the upstream face of a high curved masonry dam, reservoir pressures near the base, together with horizontal circumferential stresses and Poisson's ratio effects, may offset the calculated vertical tension stresses which might otherwise be considered detrimental. In arch dams, principal stresses along abutment locations may be more pertinent than either horizontal arch stresses or vertical cantilever stresses. In gravity dams built at canyon sites where abutment slopes are relatively steep, stresses caused by twist effects may be important considerations. In unusually high masonry dams, shearing stresses may constitute a more important criterion of safety than direct stresses. Stress concentrations at boundary irregularities, around gallery openings, and at other local discontinuities may also need to be investigated carefully (27).

Local stress concentrations may be studied by theoretical analytical methods, by photoelastic tests, or by actual measurements on small laboratory models. All three methods have been used by the Bureau of Reclamation in connection with design work on masonry dams of both arch and gravity types (28) (29) (30). Whenever two or more methods of investigation were used for the same problem, results by the different methods were found to check satisfactorily.

Formulas for calculating stress conditions in the interior of a masonry dam, based on a straight-line distribution of vertical stress and a parabolic distribution of shear stress, were developed during the design work on Boulder Dam. The derivations were completed about the time that W. H. Holmes, Assoc. M. Am. Soc. C. E., pubished his paper (31). The formulas were accurately derived, using methods which were substantially the same as those subsequently suggested by W. C. Huntington, M. Am. Soc. C. E., in his discussion of Mr. Holmes' paper. It is interesting to note that shearing stresses calculated by using horizontal planes 1 ft, or even 0.1 ft, apart, as originally proposed by the late William Cain (32), M. Am. Soc. C. E., and used by Mr. Holmes, did not agree satisfactorily with those calculated by the theoretical formulas. In order to secure a satisfactorily accurate result by the two-plane method, it was necessary to take horizontal planes 0.001 ft apart. In 1937 an English engineer, Serge Leliavsky, proposed a graphical method (33) of determining shearing stresses in gravity dams which gives results substantially the same as the theoretical formulas derived by the Bureau of Reclamation.

From the viewpoint of stress conditions, the most noteworthy modern improvements in methods of masonry dam design have been the development of methods of analyzing effects of foundation and abutment deformations and methods of determining the true nonlinear distribution of stress within the structure. The analysis of foundation and abutment deformations was begun by Fredrik Vogt, Assoc. M. Am. Soc. C. E., in 1925 and was further developed by engineers of the Bureau of Reclamation in connection with the work on Owyhee (in Oregon) and Boulder dams (34). Methods of determining the true nonlinear distribution of stress within the dam, including the evaluation of shearing stresses along both horizontal and vertical planes, as well as the evaluation of direct stresses in all directions, were developed by the engineers of the Bureau in connection with the design work on Boulder and Grand Coulee dams, the investigation being conducted by both laboratory and analytical methods. Determinations of nonlinear stress conditions have shown that, although the linear assumption may not be greatly in error in the upper central portions of the dam, it usually is not sufficiently correct for locations near the foundations and abutments. Comparisons of calculated interior stresses based on an assumption of a linear distribution, with nonlinear stresses determined by model measurements, are given in the bulletin on "Slab Analogy Experiments," published in 1938 as one of the final reports on the Boulder Canyon Project (35). Similar comparisons in which nonlinear stresses were determined by trial load methods were published in a subsequent bulletin of the same series (36).

The trial load method of determining nonlinear stresses in the interior of a masonry dam is based on the adjustment of horizontal and vertical beam deflections at all conjugate points in the interior of the dam and in a large foundation block which is included in the analysis as an integral part of the structure. Results obtained by such analyses agree very satisfactorily with those secured by measurements on laboratory models. Although the Bureau's investigations of nonlinear stress conditions were begun in connection with the design of Boulder Dam, a massive arched structure, similar investigations have also been made for straight gravity dams such as Norris (in Tennessee) and Grand Coulee. Drawings showing nonlinear stress conditions determined by measurements made on slab models of Boulder, Norris, and Grand Coulee dams were published in 1934 and 1938 (37) (38).

SUMMARY

The basic assumptions involved in analyzing stresses in masonry dams have been listed in preceding subdivisions of this paper. Summarizing briefly, the more important assumptions are numbers 3, 6, 7, 8, and 10. In other words, the fundamental assumption made in analyzing stresses is that the dam is a homogeneous, uniformly elastic structure, built on a uniformly elastic foundation, and abutting against uniformly elastic canyon walls.

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DESIGN OF ARCH DAMS

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Synopsis

The purpose of this paper is to present a comprehensive series of tables to facilitate the computation of forces, moments, and radial deflections in the design of arch dams. Seven basic load conditions are thus provided for, in an Appendix, and the text comprises a brief discussion of design problems involved.

General

An ideal arch dam may be defined as a structure by which the total water load is transmitted to the abutments by arch action. The abutments may be either the natural walls of a canyon or they may be constructed artificially. This ideal condition is seldom realized, and a part of the water load is usually carried by the gravity elements of the structure.

The gravity elements and vertical beam action introduce a complexity in the design, and a true or nearly true arch dam is sometimes produced by the introduction of an articulated joint at the base of the dam. This may be permissible in exceptional cases in which it is impossible to design for the high shears found at the base, and if it is possible to develop an articulated joint that is 100% efficient throughout its entire length. The failure of this joint to operate at any point will introduce a discontinuity in the structure with consequent secondary stresses of undetermined magnitude. It is the opinion of the writer that even if the ideal conditions can be met for the articulated structure, it is not desirable to destroy any part of the structural strength of the dam in order to simplify its analysis.

The ideal site for an arch dam is a canyon having a small ratio of length to height and having reasonably sound abutments which are so nearly symmetrical that symmetry can be produced with a small amount of excavation.

An economical and relatively simple structure will result when the ratio of length to height does not exceed about 2½ to 1. It will be found that a considerable proportion of the water load is carried by gravity action when this ratio reaches 5 to 1. At about that value, the arch will become quite massive, and its additional length and high unit costs make its selection doubtful unless it is possible to take advantage of topographic conditions in the foundation, as in the case where the narrow part of the canyon has been scoured to a greater depth than the section upstream. In that case it may be economical to take advantage of this condition with the curved structure. It should be noted that the ratios given are relative only, and that local conditions will alter them considerably.

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The effect of lack of symmetry or lack of continuity cannot be predicted, but must be determined by analysis. These effects may be so great as to preclude the use of an arch dam, unless it is found feasible to construct an abutment or thrust block which produces the required symmetry.

These conditions affect only the economy of the structure, as it is possible to design for any reasonable condition, but the work of design becomes much more difficult, and should not be attempted until the designer has had considerable experience in the design of arch dams.

The conditions that must be met at the abutments are complex, and the most severe loading conditions may not occur at the time of maximum stress in the dam, which usually occurs with maximum water load and minimum temperature of the concrete. At this time, the resultant force applied to the abutment will have its most dangerous direction; but usually it will have its minimum magnitude in the top of the abutment.

This condition of minimum magnitude may not hold in special cases. There are cases in which the water load is transferred by tangential shear toward the upper part of the abutment. This may be caused by unusual topographic conditions, or by the necessity of building the upper portions of the dam to a greater than normal thickness to provide for roadway or ice load.

It follows that the maximum abutment load will occur in normal cases with full water load and maximum concrete temperatures. This is not the most critical case for natural abutments, as the shear produced by the plus temperature in the arches is toward the upstream face at the abutment, which throws the resultant farther back into the hill. This condition must be considered when artificial abutments are necessary, as the lateral stability of the abutment must be considered.

The magnitude and variation in direction of the resultant forces must be computed, and sufficient mass must be provided in the abutment to take the most severe conditions with a high factor of safety. Due to the usual jointing in the rock and possible presence of hydrostatic pressure in the abutment, and the usual presence of tension in the abutment at the intersection of the upstream face and abutment, it is advisable to consider as acting only that part of the abutment which comes under direct bearing from the arch.

It should be noted that the angle of intersection of the arch tangent and the abutment is not a criterion for judging the direction of the applied force. The arch thrust is only one component of the true force, and is separated for convenience in computation. The arch shear is the other force component. There are many cases in which the resultant abutment force is directed out of the abutment toward the river. This will occur with thick arches having a small central angle and a minimum temperature condition. It will also occur with thin arches having a central angle close to 180°.

ANALYSIS

The structure to be analyzed is inherently complex, as is the distribution of forces throughout the structure. This distribution is most clearly defined by the methods of the theory of elasticity, but as there is no known formal method of solution, the effects of the various loads must be evaluated by the methods of ordinary strength of materials; and the final solution must be approached by trial.

The general conditions imposed by the theory of elasticity are that each element in the dam must be in static equilibrium, and that the continuity of the structure is undisturbed by loading. It also follows that the state of stress throughout the structure is determined by the state of stress on any mutually perpendicular coordinate system. The obvious system to follow is a modified system of cylindrical coordinates, in which vertical planes will cut radial slices through the dam, called cantilevers, and horizontal planes will cut horizontal arches. This method must be considered as the one giving the greatest ease of solution, as the sections cut from the dam by planes spaced at unit distance have the characteristics of members that can be treated by the ordinary mechanics of materials. This arbitrary selection of a coordinate system in no way affects the distribution of load within the structure, but gives a means of evaluating vertical and horizontal components of stress and strain; and, as the problem is determinate from these two sets of stresses, the true distribution of load and method by which it is carried to the abutments can be evaluated.

The validity of the method has been demonstrated by the remarkable accuracy with which field and laboratory measurements have been checked by analysis.

The trial-load method of analyzing dams, in its present state of development, is the result of many years of study and research at the U. S. Bureau of Reclamation, having as its objective the development of a method that would be (1) workable and (2) capable of meeting the rigorous test of laboratory and field measurements. The earlier works in this field, notably that of H. E. Gumer, Professor Rohn, and Albert Stukie in Europe, and the work of Julian Hinds and C. H. Howell, Members, Am. Soc. C. E., and the late A. C. Jaquith, in the United States, have paved the way for the present methods.

Later developments in the method have been made possible by contributions from Fredrik Vogt, Assoc. M. Am. Soc. C. E., H. M. Westergaard and Ivan E. Houk, Members, Am. Soc. C. E. To Robert E. Glover, engineer, Bureau of Reclamation, goes the credit of having made the first trial-load analysis that includes the effect of torsion, tangential shear, and Poisson's ratio.[5] The method as presented by Mr. Glover represents the greatest refinement found practicable in arch dam analysis.

CHOICE OF SECTION FOR DAM

The choice of the section for a given dam site depends entirely on its individual characteristics. The most suitable section may not be found without careful study and alternate designs which are selected for the purpose of evaluating these characteristics. Certain fundamentals will be of assistance in making the layout.

The economical central angle of an arch is one regarding which there are many misconceptions. Publications often give this angle as 120°, a value that results from the thin cylinder formula without regard for temperature. The elastic theory gives a value of approximately 150° to 160°; and special cases in which high temperature stresses are used give values close to 180° for the economical central angle.

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[5] *Bulletin No. 1*, Part V, Technical Investigations, Boulder Canyon Final Reports, Bureau of Reclamation, U. S. Dept. of the Interior.

It should be noted that the most economical central angle can be obtained only for one set of working conditions; and the most economical angle seldom can be used, due to the abutment and foundation conditions, as the angle of the resultant at the abutment may be of greater importance.

The zone of higher stress is likely to occur at about two thirds of the height of the structure, at which point most of the load is carried by arch action. The examination of a given section at this point, using the full water load, may disclose a weakness or overdesign in the structure and eliminate the necessity of making a detailed analysis of the entire dam.

In order that a suitable section may be more readily arrived at by a small organization lacking the time and personnel to make extensive preliminary design studies there is presented herein a tabulation of constants, the use of which will eliminate a considerable number of the calculations necessary to complete a radial adjustment. The only calculations necessary to obtain all the arch data required for the radial adjustment can be made with a slide rule in a short time. The value of such data is best illustrated by quoting from the Reclamation Bureau bulletin on "The Trial Load Method of Analyzing Arch Dams":[5]

> "The radial adjustment constitutes the most important step in the trial load analysis. Although tangential and twist adjustments are usually advisable, they are of secondary importance when compared with the radial adjustment. The reason for this is evident from the fact that radial movements are much larger than tangential or angular movements. An analysis based on the adjustment of radial deflections only is often made as a preliminary study. Such an analysis has been found to give a fair indication of maximum stresses. This type of analysis is used to compare preliminary designs for a dam, the complete analysis being used to determine stresses in the adopted design."

TABULAR COEFFICIENTS AND FORMULAS FOR DESIGNING CIRCULAR ARCHES

The Appendix includes fourteen tables for use in designing circular arches, two being necessary for each load. For any load Table (*a*), of a pair, lists the factors necessary to obtain the forces and moments at the crown and the abutment of the arch, whereas Table (*b*) of that pair lists the factors necessary to obtain the deflections of the quarter points of the arch. For the most part the values are to four significant figures; however, in some cases only three significant figures are used. Some of the tabular values have several zeros following the decimal point. In such cases the number of zeros between the decimal point and the first significant figure is designated by a superior number; for example, 0.0000154 is written 0.0^4154.

The unit loads listed are as follows: No. 1, a uniform radial load (Table 3, Appendix); Nos. 2, 3, 4, and 5, triangular radial loads (Tables 4, 5, 6, and 7, Appendix); temperature load, consisting of a unit temperature change of minus 10° (Table 8, Appendix); and concentrated radial load, applied at the abutment (Table 9, Appendix). By means of these loads it is possible to produce any load that varies as a straight line between quarter points. Each load is illustrated on its respective sheet in the tables.

The following definitions as to signs are used throughout the tables:

1. A positive thrust (H) produces compression;
2. A positive moment (M) tends to produce compression in the extrados of the arch;
3. A positive shear (V) tends to move a section of the arch upstream with respect to a section nearer the abutment;
4. A positive radial load acts downstream toward the center of the arch;
5. A positive deflection is a movement upstream; and
6. A temperature rise is assumed positive.

Inasmuch as the values listed in Tables 3 to 9 (Appendix) include the effects of bending, ribshortening, shear, and abutment deformations, certain assumptions over the ordinary were necessary in their calculations. These are as follows:

The modulus of elasticity of the concrete in tension and compression (E_c) is equal to the modulus of elasticity of the foundation and abutment rock (E_R). This assumption is valid for most cases. If conditions are encountered which differ greatly from this, independent calculation of the arch data will be necessary using the constants

$$\alpha = \frac{5.075}{E_R\, t^2} \qquad \text{(1a)}$$

$$\beta = \frac{1.556}{E_R} \qquad \text{(1b)}$$

and

$$\gamma = \frac{1.785}{E_R} \qquad \text{(1c)}$$

in which: t = thickness of the arch ring; α = average rotation in a plane normal to the foundation surface due to the unit moment load; β = average deformation normal to the foundation surface due to the normal unit load; and γ = average deformation in the plane of the foundation surface due to unit shear load. In an adjustment using Tables 3 to 9 (Appendix) the factors expressed by Eqs. 1 must be used also in computing the cantilever deflections.

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This work has been prepared under the supervision of J. L. Savage, M. Am. Soc. C. E., chief designing engineer. All construction and engineering of the Department of the Interior, Bureau of Reclamation, is under the general supervision of R. F. Walter, M. Am. Soc. C. E., chief engineer with headquarters in Denver, Colo., and all activities of the Bureau are under the general charge of

John C. Page, M. Am. Soc. C. E., commissioner, with headquarters in Washington, D. C.

APPENDIX

The Use of the Tables

The caption of each table describes the load and the type of values to be found on that sheet (forces and moments, or deflections). In any table the extreme left column is designated ϕ_A and represents the half central angle of the arch. At the top of the tables are listed the values $\frac{t}{r}$, in which t is the arch thickness and r is the radius of the arch center line. Straight-line interpolation between the values $\frac{t}{r}$ and ϕ_A to obtain the desired value from the table will be sufficiently accurate for most cases.

For example, consider a 63° arch with a radius of 500 ft and a thickness of 40 ft: $E_c = 4{,}000{,}000$ lb per sq in. $= 576{,}000{,}000$ lb per sq ft; and $\frac{t}{r} = 0.080$. The thrusts at the crown and the quarter points, due to load No. 1, are interpolated as shown in Table 1. From Table 1(*a*), $h = 1{,}024$; and $H = 1{,}024 \times 500 = 512{,}000$ lb. From Table 1(*b*), $K_1 = -23{,}210$; and

$$\Delta = \frac{-23{,}210 \times 500}{576{,}000{,}000} = -0.02035.$$

TABLE 1.—Use of Tables 3 to 9; Illustrative Example

Values of ϕ_A, in degrees	Values of $\frac{t}{r}$					
	0.075	0.080	0.100	0.075	0.080	0.100
	(*a*) Thrust at the Crown Due to Load No. 1			(*b*) Deflection at the Quarter Point Due to Load No. 1		
60	+1,022	+1,022	+1,022	−24,560	−23,440	−18,950
63	......	+1,024	......	......	−23,210	......
70	+1,030	+1,031	+1,035	−24,470	−23,360	−18,900

The variety of sections which are possible for cantilevers makes tabulations such as those presented for arches impossible.[6]

Radial Load Description.—Referring to Fig. 1, P is the total load at the abutment—in pounds per square feet when the load is distributed and in pounds when the load is concentrated. The loads indicated by the diagrams in Tables

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[6] To compute cantilever stresses and deflections, refer to *Bulletin No. 1*, Part V, Technical Investigations, Boulder Canyon Final Reports, Bureau of Reclamation, U. S. Dept. of the Interior, Chapter IV, p. 63.

3 to 9 are unit radial loads applied at the upstream face (except the unit temperature load) and may be combined into any pattern that varies as a straight line between quarter points. The various loads are described in Table 2. The unit load shown in the diagrams in Tables 3 to 9 (Appendix) results in forces or moments determined by the corresponding formulas given in the tables.

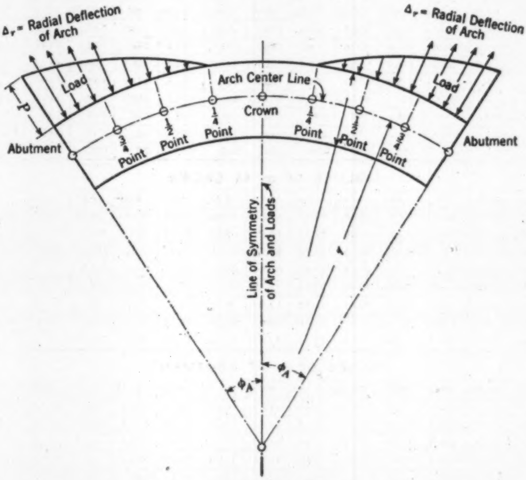

Δ_r = Radial Deflection of Arch
Δ_r = Radial Deflection of Arch
Load
Load
P
Arch Center Line
Crown
$\frac{1}{4}$ Point
$\frac{1}{2}$ Point
$\frac{3}{4}$ Point
$\frac{1}{4}$ Point
$\frac{1}{2}$ Point
$\frac{3}{4}$ Point
Abutment
Abutment
Line of Symmetry of Arch and Loads
ϕ_A
ϕ_A

FIG. 1.—PROPERTIES OF A CIRCULAR ARCH

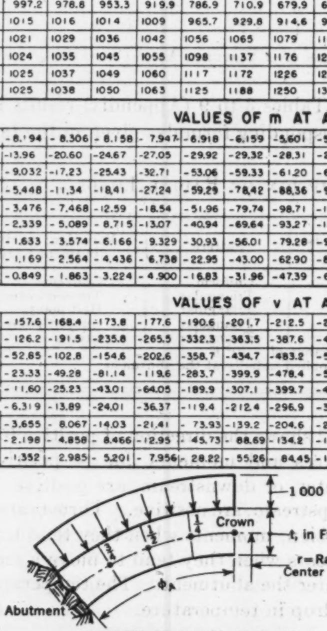
TABLE 2.—DESCRIPTION OF RADIAL LOADS

Table No.	Load No.	Intensity, in pounds per square foot	Pattern	Variation; maximum at abutment to zero at:
3	1	10,000	Uniform	Uniform
4	2	10,000	Triangular	Three-quarter point
5	3	10,000	Triangular	Half point
6	4	10,000	Triangular	Quarter point
7	5	10,000	Triangular	Crown
8	Temperature	Uniform change of −10° F		
9	Concentrated	1,000 lb, applied radially at the abutment		

Positive Sign Conventions.—Fig. 2 shows the direction of positive forces and moments, and the direction of forces and moments due to positive loads. Loads directed toward the arch center, or downstream, are positive. Deflections away from the arch center, or upstream, are positive. Thrusts are positive when they tend to produce compression; moments when they tend to produce compression in the extrados; and shears when they tend to move a section upstream with respect to a section nearer the abutment. The temperature table (Table 8, Appendix) is based on a drop in temperature.

TABLE 3(*a*).—Forces and Moments for Radial Load No. 1

ϕ_A	VALUES OF t/r															
	.025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.300
VALUES OF h AT CROWN																
10	310.1	117.5	67.91	48.84	39.69	27.65	26.22	26.46	27.20	28.14	29.18	30.28	31.41	32.56	33.73	34.91
20	880.6	656.0	477.5	360.7	286.2	153.4	124.8	116.8	115.4	116.6	119.2	122.5	126.2	130.2	134.4	138.7
30	983.1	919.3	831.9	740.8	657.4	407.6	318.0	283.7	270.7	267.4	269.1	273.6	279.8	287.1	295.1	303.7
40	1003	988.7	960.8	923.8	876.4	683.7	565.3	505.7	477.7	466.7	465.4	469.8	477.8	488.3	500.4	513.6
50	1009	1010	1005	993.8	978.9	877.1	786.6	728.3	696.4	682.4	679.9	685.6	696.3	710.6	727.4	746.1
60	1011	1018	1022	1022	1021	987.2	942.3	907.2	886.8	879.5	882.5	893.0	909.0	929.1	952.1	977.5
70	1012	1021	1029	1035	1039	1046	1039	1032	1031	1038	1056	1072	1096	1125	1156	1189
80	1012	1023	1033	1041	1049	1079	1097	1114	1132	1155	1182	1213	1249	1287	1328	1371
90	1012	1024	1035	1045	1055	1097	1132	1166	1200	1237	1277	1320	1366	1415	1466	1519
VALUES OF m AT CROWN																
10	3.967	5.593	6.424	7.052	7.592	9.753	11.48	12.99	14.35	15.62	16.83	17.98	19.10	20.19	21.26	22.31
20	2.817	8.293	13.18	16.90	19.77	28.68	34.77	40.02	44.86	49.44	53.83	58.08	62.23	66.30	70.29	74.23
30	1.377	5.128	10.31	15.99	21.56	43.05	57.16	68.27	78.03	87.07	95.67	104.0	112.1	120.1	127.9	135.6
40	0.775	3.042	6.593	11.12	16.30	43.96	67.14	85.76	101.6	115.9	129.2	141.8	154.0	165.9	177.6	189.1
50	0.486	1.935	4.295	7.470	11.34	36.58	63.47	87.66	109.9	128.0	145.6	162.0	177.7	192.8	207.5	222.0
60	0.328	1.310	2.928	5.150	7.926	27.98	52.98	78.15	101.7	123.4	143.4	162.1	179.8	196.7	213.1	228.9
70	0.232	0.926	2.075	3.661	5.664	20.84	41.48	63.99	86.29	107.5	126.0	146.0	163.5	180.1	196.0	217.2
80	0.170	0.675	1.512	2.668	4.131	15.42	31.42	49.67	68.39	86.58	103.8	119.9	134.9	148.9	162.1	174.6
90	0.127	0.503	1.122	1.977	3.056	11.39	23.31	37.07	51.29	65.09	77.97	89.73	100.3	109.9	118.4	126.0
VALUES OF h AT ABUTMENT																
10	320.8	131.3	82.64	64.06	55.23	44.32	43.86	45.05	46.73	48.60	50.58	52.61	54.67	56.76	58.85	60.96
20	888.6	678.3	511.2	402.3	333.0	212.0	188.9	185.2	187.6	192.5	198.7	205.5	212.8	220.3	228.0	235.9
30	987.0	933.5	859.5	782.3	711.7	503.7	434.5	413.1	410.3	415.8	425.6	437.9	451.7	466.3	481.7	497.4
40	1005	997.2	978.8	953.3	919.9	786.9	710.9	679.9	673.0	679.2	692.8	710.8	731.6	754.2	778.1	802.9
50	1010	1015	1016	1014	1009	965.7	929.8	914.6	916.5	929.8	950.6	976.5	1006	1037	1070	1105
60	1012	1021	1029	1036	1042	1056	1065	1079	1100	1127	1160	1196	1236	1277	1320	1364
70	1012	1024	1035	1045	1055	1098	1137	1176	1216	1260	1307	1353	1403	1454	1506	1558
80	1012	1025	1037	1049	1060	1117	1172	1226	1281	1337	1393	1450	1508	1566	1625	1684
90	1013	1025	1038	1050	1063	1125	1188	1250	1313	1375	1438	1500	1563	1625	1688	1750
VALUES OF m AT ABUTMENT																
10	-6.704	-8.194	-8.306	-8.158	-7.947	-6.918	-6.159	-5.601	-5.175	-4.839	-4.569	-4.346	-4.160	-4.001	-3.865	-3.746
20	-5.136	-13.96	-20.60	-24.67	-27.05	-29.92	-29.32	-28.31	-27.33	-26.45	-25.68	-24.99	-24.39	-23.85	-23.38	-22.95
30	-2.563	-9.032	-17.23	-25.43	-32.71	-53.06	-59.33	-61.20	-61.55	-61.32	-60.86	-60.31	-59.74	-59.19	-58.65	-58.14
40	-1.447	-5.448	-11.34	-18.41	-27.24	-59.29	-78.42	-88.36	-93.66	-96.60	-98.28	-99.24	-99.78	-100.1	-100.2	-100.2
50	-0.903	-3.476	-7.468	-12.59	-18.54	-51.96	-79.74	-98.71	-110.1	-119.4	-125.0	-128.9	-131.8	-133.8	-135.4	-136.6
60	-0.602	-2.339	-5.089	-8.715	-13.07	-40.94	-69.64	-93.27	-111.2	-124.4	-134.1	-141.4	-146.9	-151.2	-154.6	-157.3
70	-0.419	-1.633	-3.574	-6.166	-9.329	-30.93	-56.01	-79.28	-98.75	-114.3	-125.0	-135.9	-143.3	-149.2	-153.8	-157.6
80	-0.299	-1.169	-2.564	-4.436	-6.738	-22.95	-43.00	-62.90	-80.60	-95.42	-107.4	-116.9	-124.4	-130.3	-134.9	-138.5
90	-0.217	-0.849	-1.863	-3.224	-4.900	-16.83	-31.96	-47.39	-61.42	-73.30	-82.87	-90.29	-95.87	-99.92	-102.7	-104.6
VALUES OF v AT ABUTMENT																
10	-122.0	-157.6	-168.4	-173.8	-177.6	-190.6	-201.7	-212.5	-223.2	-233.9	-244.6	-255.2	-265.9	-276.5	-287.2	-297.8
20	-45.11	-126.2	-191.5	-235.8	-265.5	-332.3	-363.5	-387.6	-409.4	-430.4	-450.9	-471.1	-491.2	-511.3	-531.2	-551.1
30	-14.70	-52.85	-102.8	-154.6	-202.6	-358.7	-434.7	-483.2	-520.9	-553.8	-584.2	-613.2	-641.3	-669.0	-696.2	-723.2
40	-6.104	-23.33	-49.28	-81.14	-119.6	-283.7	-399.9	-478.4	-536.6	-583.8	-624.9	-662.2	-697.2	-730.7	-763.1	-794.7
50	-2.978	-11.60	-25.23	-43.01	-64.05	-189.9	-307.1	-399.7	-472.0	-530.6	-580.3	-623.9	-663.6	-700.5	-735.5	-769.1
60	-1.612	-6.319	-13.89	-24.01	-36.37	-119.4	-212.4	-296.9	-368.7	-429.1	-480.7	-525.7	-565.9	-602.7	-636.8	-669.0
70	-0.929	-3.655	-8.067	-14.03	-21.41	-73.93	-139.2	-204.6	-264.3	-316.7	-358.5	-402.6	-438.2	-470.2	-499.6	-526.7
80	-0.559	-2.198	-4.858	-8.466	-12.95	-45.73	-88.69	-134.2	-177.6	-216.9	-251.7	-282.2	-309.0	-332.8	-354.0	-373.1
90	-0.344	-1.352	-2.985	-5.201	-7.956	-28.22	-55.26	-84.45	-112.7	-138.4	-160.8	-180.0	-196.2	-209.8	-221.1	-230.6



Thrust, Moment, Shear, in Lb, and Ft-Lb

$H = hr$

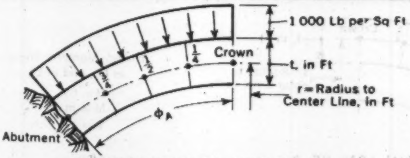
$M = mr^2$

$V = vr$

DIAGRAM OF RADIAL LOAD NO. 1

TABLE 3(*b*).—RADIAL DEFLECTIONS FOR RADIAL LOAD NO. 1

ϕ_A	VALUES OF t/r															
	.025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
	VALUES OF K_1 AT CROWN															
10	-27190	-5956	-2626	-1604	-1168	-628.7	-538.2	-511.8	-507.1	-511.5	-520.6	-532.5	-546.2	-561.1	-576.8	593.2
20	-71560	-28760	-15020	-9141	-6208	-2256	-1557	-1327	-1232	-1193	-1180	-1183	-1194	-1212	-1233	-1258
30	-77910	-38350	-28330	-17070	-12720	-4936	-3118	-2462	-2169	-2022	-1949	-1914	-1905	-1910	-1927	-1949
40	-78590	-40260	-27090	-20280	-16070	-7428	-4804	-3716	-3186	-2903	-2745	-2657	-2612	-2595	-2597	-2612
50	-78550	-40560	-27720	-21200	-17210	-8900	-6085	-4779	-4061	-3695	-3463	-3324	-3244	-3202	-3187	-3191
60	-78510	-40540	-27820	-21400	-17530	-9560	-6823	-5485	-4737	-4293	-4017	-3846	-3741	-3682	-3654	-3648
70	-78460	-40490	-27780	-21400	-17560	-9798	-7168	-5869	-5126	-4671	-4374	-4199	-4082	-4015	-3980	-3969
80	-78540	-40470	-27730	-21350	-17520	-9849	-7289	-6033	-5313	-4867	-4582	-4397	-4280	-4209	-4161	-4160
90	-78730	-40480	-27700	-21310	-17470	-9821	-7299	-6071	-5369	-4935	-4657	-4476	-4360	-4290	-4254	-4241
	VALUES OF K_1 AT $\frac{1}{4}$ POINT															
10	-24270	-5403	-2418	-1496	-1100	-608.6	-525.5	-503.2	-500.2	-505.5	-515.3	-527.7	-541.7	-556.8	-572.9	-588.3
20	-63370	-25680	-13520	-8290	-5680	-2124	-1490	-1283	-1199	-1165	-1156	-1160	-1173	-1192	-1215	-1240
30	-68780	-34040	-21710	-15310	-11470	-4558	-2930	-2341	-2041	-1949	-1885	-1858	-1853	-1862	-1874	-1906
40	-69230	-35610	-24060	-18080	-14390	-6777	-4452	-3485	-3015	-2764	-2626	-2552	-2516	-2505	-2511	-2530
50	-69080	-35790	-24550	-18830	-15330	-8054	-5581	-4431	-3806	-3479	-3278	-3159	-3092	-3060	-3052	-3061
60	-68960	-35700	-24560	-18950	-15560	-8602	-6210	-5042	-4332	-4002	-3765	-3619	-3532	-3485	-3468	-3466
70	-68800	-35580	-24470	-18900	-15550	-8777	-6485	-5356	-4712	-4319	-4062	-3916	-3826	-3765	-3740	-3737
80	-68740	-35470	-24370	-18810	-15470	-8790	-6565	-5476	-4853	-4469	-4225	-4069	-3972	-3915	-3887	-3883
90	-68730	-35400	-24280	-18720	-15390	-8338	-6548	-5486	-4880	-4507	-4268	-4115	-4019	-3982	-3934	-3928
	VALUES OF K_1 AT $\frac{1}{2}$ POINT															
10	-16520	-3903	-1843	-1193	-908.9	-550.2	-490.8	-477.7	-479.4	-487.6	-499.3	-513.1	-528.2	-544.1	-560.6	-577.7
20	-41800	-17460	-9474	-5982	-4214	-1749	-1299	-1153	-1099	-1081	-1082	-1094	-1112	-1135	-1160	-1188
30	-44840	-22660	-14750	-10610	-8106	-3513	-2401	-1996	-1819	-1737	-1702	-1694	-1702	-1720	-1746	-1777
40	-44800	-23420	-16070	-12270	-9906	-5002	-3474	-2835	-2526	-2366	-2284	-2246	-2235	-2243	-2261	-2290
50	-44420	-23330	-16210	-12590	-10380	-5774	-4202	-3486	-3072	-2868	-2749	-2685	-2657	-2651	-2663	-2685
60	-44100	-23100	-16080	-12540	-10410	-6040	-4548	-3823	-3422	-3190	-3054	-2976	-2936	-2922	-2927	-2945
70	-43730	-22860	-15890	-12400	-10300	-6067	-4651	-3962	-3575	-3344	-3195	-3123	-3081	-3060	-3061	-3076
80	-43420	-22640	-15710	-12240	-10150	-6001	-4633	-3972	-3601	-3378	-3241	-3159	-3113	-3092	-3085	-3101
90	-43110	-22430	-15530	-12080	-10030	-5905	-4563	-3919	-3557	-3340	-3205	-3122	-3074	-3050	-3044	-3051
	VALUES OF K_1 AT $\frac{3}{4}$ POINT															
10	-4869	-1931	-1054	-760.9	-628.8	-458.0	-434.8	-435.9	-445.2	-458.1	-473.0	-489.0	-505.7	-522.9	-540.5	-558.4
20	-15790	-7256	-4305	-2954	-2243	-1201	-1005	-950.3	-939.1	-946.6	-963.7	-968.3	-1012	-1041	-1071	-1101
30	-16350	-8833	-6122	-4676	-3750	-2064	-1629	-1476	-1420	-1406	-1414	-1433	-1461	-1494	-1530	-1568
40	-16000	-8820	-6359	-5084	-4292	-2630	-2104	-1890	-1798	-1763	-1759	-1773	-1798	-1830	-1868	-1910
50	-15630	-8586	-6218	-5022	-4296	-2810	-2323	-2109	-2006	-1968	-1959	-1969	-1992	-2023	-2060	-2102
60	-15330	-8352	-6029	-4865	-4169	-2782	-2344	-2153	-2063	-2025	-2016	-2026	-2047	-2077	-2113	-2153
70	-15020	-8142	-5851	-4708	-4026	-2684	-2260	-2095	-2012	-1978	-1960	-1978	-1996	-2023	-2055	-2091
80	-14740	-7952	-5694	-4566	-3893	-2576	-2171	-1997	-1915	-1878	-1866	-1870	-1883	-1904	-1928	-1958
90	-14460	-7778	-5552	-4442	-3778	-2476	-2072	-1895	-1807	-1764	-1734	-1740	-1745	-1757	-1773	-1792
	VALUES OF K_1 AT ABUTMENT															
10	-217.7	-261.3	-300.5	-310.3	-317.0	-340.1	-360.0	-379.3	-398.4	-417.5	-436.5	-455.6	-474.6	-493.6	-512.6	-531.6
20	-80.51	-225.3	-341.9	-420.8	-473.9	-593.2	-648.8	-691.8	-730.9	-768.3	-804.8	-841.0	-876.9	-912.6	-948.2	-983.7
30	-26.25	-94.33	-183.5	-275.9	-361.6	-640.2	-776.0	-862.5	-929.8	-988.5	-1043	-1095	-1145	-1194	-1243	-1291
40	-10.90	-41.64	-87.97	-144.8	-207.5	-506.4	-713.9	-853.9	-957.8	-1042	-1115	-1182	-1245	-1304	-1362	-1419
50	-5.316	-20.71	-45.03	-76.77	-114.3	-338.9	-548.2	-713.4	-842.5	-947.1	-1036	-1114	-1184	-1250	-1313	-1373
60	-2.877	-11.28	-24.79	-42.87	-64.92	-213.1	-379.1	-530.0	-658.1	-765.9	-858.0	-938.4	-1010	-1076	-1137	-1194
70	-1.658	-6.524	-14.40	-25.06	-38.22	-132.0	-248.5	-365.2	-471.7	-565.3	-639.9	-718.6	-782.1	-839.4	-802.5	-940.2
80	-0.9976	-3.923	-8.672	-15.11	-23.12	-81.63	-158.3	-239.5	-317.0	-387.2	-449.3	-503.8	-551.6	-594.0	-631.8	-666.0
90	-0.6140	-2.413	-5.328	-9.284	-14.20	-50.38	-98.64	-150.7	-201.2	-247.0	-287.1	-321.3	-350.2	-374.4	-394.7	-411.6



Radial Deflection, in Ft

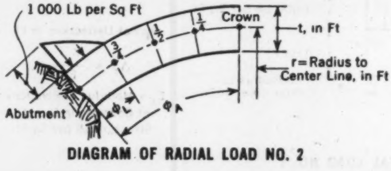
$$\Delta_r = \frac{K_1 r}{E_c}$$

E_C = Modulus of Elasticity of Concrete in Direct Stress, in Lb per Sq Ft

DIAGRAM OF RADIAL LOAD NO. 1

TABLE 4(*a*).—Forces and Moments for Radial Load No. 2

ϕ_A	VALUES OF t/r															
	.025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
	VALUES OF h AT CROWN															
10	6.046	3.198	2.885	2.873	2.949	3.477	3.924	4.302	4.641	4.954	5.252	5.541	5.822	6.097	6.369	6.637
20	10.28	10.29	9.976	9.850	9.921	11.42	13.14	14.71	16.13	17.46	18.71	19.92	21.08	22.21	23.32	24.43
30	10.44	11.94	13.14	14.18	15.16	19.68	23.75	27.38	30.67	33.70	36.56	39.29	41.92	44.46	46.95	49.36
40	10.22	11.75	13.25	14.74	16.25	23.99	31.33	37.93	43.88	49.35	54.45	59.29	63.91	68.38	72.72	76.95
50	10.08	11.46	12.85	14.30	15.78	24.53	34.15	43.60	52.48	60.77	68.54	75.94	82.98	89.76	96.33	102.7
60	10.05	11.27	12.52	13.82	15.21	23.51	33.70	44.72	55.81	66.60	76.97	86.93	96.51	105.8	114.7	123.5
70	10.09	11.19	12.32	13.49	14.74	22.24	31.95	43.23	55.33	67.68	79.98	92.06	103.9	115.4	126.6	137.6
80	10.19	11.20	12.23	13.30	14.43	21.16	30.01	40.75	52.85	65.76	79.08	92.52	105.9	119.2	132.3	145.3
90	10.35	11.28	12.24	13.23	14.27	20.34	28.30	38.18	49.69	62.39	75.90	89.90	104.2	118.6	133.0	147.3
	VALUES OF m AT CROWN															
10	-.0152	.0336	.0489	.0615	0.727	0.118	0.153	0.183	0.209	0.234	0.256	0.278	0.298	0.318	0.338	0.357
20	-0.104	-.0803	-.0496	-.0233	$-.0^{2}21$	.0633	0.105	0.141	0.172	0.202	0.230	0.257	0.283	0.308	0.333	0.483
30	-0.255	-0.290	-0.315	-0.336	-0.356	-0.475	-0.611	-0.749	-0.885	-1.017	-1.145	-1.271	-1.395	-1.516	-1.636	-1.754
40	-0.441	-0.517	-0.596	-0.681	-0.773	-1.330	-1.960	-2.594	-3.214	-3.820	-4.413	-4.995	-5.569	-6.136	-6.697	-7.253
50	-0.669	-0.776	-0.892	-1.022	-1.169	-2.162	-3.486	-4.897	-6.365	-7.832	-9.284	-10.73	-12.16	-13.57	-14.98	-16.38
60	-0.962	-1.075	-1.221	-1.384	-1.569	-2.870	-4.767	-7.036	-9.576	-12.19	-14.86	-17.54	-20.23	-22.92	-25.61	-28.30
70	-1.259	-1.418	-1.590	-1.781	-1.994	-3.502	-5.813	-8.818	-12.32	-16.14	-20.16	-24.32	-28.55	-32.83	-37.15	-41.50
80	-1.620	-1.804	-2.001	-2.217	-2.468	-4.106	-6.677	-10.17	-14.44	-19.30	-24.59	-30.21	-36.04	-42.04	-48.17	-54.39
90	-2.024	-2.232	-2.454	-2.693	-2.955	-4.714	-7.436	-11.22	-15.99	-21.62	-27.94	-34.80	-42.09	-49.71	-57.59	-65.68
	VALUES OF h AT ABUTMENT															
10	6.275	3.475	3.170	3.163	3.241	3.782	4.241	4.633	4.987	5.315	5.629	5.932	6.229	6.520	6.807	7.091
20	10.94	10.97	10.69	10.59	10.67	12.16	13.86	15.41	16.83	18.15	19.41	20.62	21.79	22.93	24.06	25.17
30	11.93	13.26	14.34	15.28	16.16	20.25	23.96	27.28	30.30	33.11	35.77	38.31	40.76	43.14	45.47	47.76
40	12.96	14.20	15.41	16.61	17.83	24.08	30.02	35.39	40.27	44.78	49.00	53.02	56.88	60.62	64.26	67.82
50	14.49	15.48	16.47	17.50	18.55	24.67	31.35	37.92	44.12	49.94	55.43	60.68	65.70	70.56	75.27	79.88
60	16.55	17.30	18.07	18.87	19.70	24.56	30.37	36.59	42.85	48.95	54.85	60.54	66.04	71.38	76.57	81.65
70	19.12	19.69	20.27	20.86	21.48	25.02	29.30	34.13	39.23	44.43	49.60	54.70	59.71	64.62	69.43	74.16
80	22.21	22.63	23.06	23.50	23.95	26.38	29.18	32.31	36.17	39.17	42.75	46.34	49.93	53.50	57.04	60.55
90	25.82	26.14	26.46	26.78	27.10	28.69	30.29	31.88	33.47	35.07	36.66	38.26	39.85	41.44	43.04	44.63
	VALUES OF m AT ABUTMENT															
10	-0.245	-0.243	-0.236	-0.228	-0.220	-0.186	-0.164	-0.148	-0.137	-0.127	-0.120	-0.114	-0.109	-0.105	-0.101	-0.098
20	-0.769	-0.760	-0.764	-0.761	-0.752	-0.675	-0.609	-0.558	-0.520	-0.490	-0.465	-0.445	-0.429	-0.414	-0.402	-0.264
30	-1.745	-1.615	-1.515	-1.432	-1.363	-1.048	-0.817	-0.648	-0.521	-0.424	-0.348	-0.287	-0.237	-0.196	-0.161	-0.132
40	-3.183	-2.963	-2.756	-2.555	-2.357	-1.420	-0.649	-0.056	0.399	0.756	1.040	1.272	1.464	1.624	1.761	1.879
50	-5.080	-4.796	-4.513	-4.225	-3.942	-2.302	-0.684	0.783	1.993	2.994	3.823	4.526	5.120	5.631	6.074	6.462
60	-7.445	-7.111	-6.774	-6.425	-6.059	-3.922	-1.435	1.069	3.388	5.452	7.263	8.846	10.23	11.46	12.54	13.51
70	-10.29	-9.920	-9.543	-9.152	-8.741	-6.277	-3.170	0.282	3.774	7.114	10.21	13.05	15.62	17.95	20.06	21.99
80	-13.64	-13.24	-12.84	-12.42	-11.99	-9.330	-5.851	-1.723	2.737	7.291	11.73	15.97	19.96	23.68	27.13	30.34
90	-17.50	-17.09	-16.67	-16.24	-15.79	-13.06	-9.420	-4.912	0.217	5.695	11.29	16.84	22.23	27.41	32.34	37.00
	VALUES OF v AT ABUTMENT															
10	-21.04	-21.80	-22.13	-22.40	-22.66	-23.94	-25.22	-26.52	-27.82	-29.13	-30.44	-31.76	-33.07	-34.39	-35.70	-37.02
20	-40.64	-41.18	-41.83	-42.42	-42.94	-45.15	-47.29	-49.48	-51.71	-53.99	-56.28	-58.60	-60.92	-63.26	-65.61	-67.96
30	-60.95	-61.02	-61.24	-61.53	-61.86	-63.69	-65.73	-68.01	-70.45	-73.00	-75.67	-78.39	-81.16	-83.97	-86.82	-89.68
40	-81.57	-81.67	-81.80	-81.92	-82.04	-82.50	-83.23	-84.42	-86.04	-87.96	-90.12	-92.46	-94.92	-97.50	-100.1	-102.9
50	-102.3	-102.6	-102.9	-103.1	-103.4	-103.4	-102.9	-102.4	-102.4	-102.8	-103.7	-104.8	-106.2	-107.8	-109.6	-111.4
60	-123.1	-123.6	-124.2	-124.7	-125.1	-126.1	-125.4	-124.0	-122.5	-121.3	-120.4	-120.0	-119.8	-119.9	-120.3	-120.8
70	-144.0	-144.8	-145.6	-146.4	-147.2	-149.6	-149.9	-148.8	-146.9	-144.8	-142.7	-140.8	-139.2	-137.8	-136.7	-135.8
80	-164.9	-166.1	-167.2	-168.3	-169.4	-173.5	-175.6	-175.8	-174.7	-172.8	-170.5	-168.0	-165.6	-163.3	-161.2	-159.3
90	-185.9	-187.4	-188.9	-190.3	-191.7	-197.7	-201.9	-204.1	-204.7	-204.1	-202.7	-200.9	-198.7	-196.4	-194.1	-191.9



Thrust, Moment, Shear,
in Lb, and Ft-Lb

$H = hr$
$M = mr^2$
$V = vr$

DIAGRAM OF RADIAL LOAD NO. 2

TABLE 4(*b*).—RADIAL DEFLECTIONS FOR RADIAL LOAD No. 2

ϕ_A	VALUES OF t/r															
	025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
	VALUES OF K_1 AT CROWN															
10	-499.9	-114.1	-74.89	-61.34	-55.24	-48.35	-48.69	-50.24	-52.17	-54.28	-56.47	-58.72	-61.01	-63.32	-65.64	-67.98
20	227.8	-208.6	-181.2	-150.8	-131.4	-100.8	-97.09	-98.36	-101.2	-104.7	-108.6	-112.7	-116.9	-121.1	-125.5	-129.7
30	3720	331.7	-13.00	-89.20	-111.5	-122.9	-125.4	-129.6	-134.7	-140.2	-146.0	-151.9	-157.9	-163.9	-170.0	-176.1
40	12580	1719	5048	183.7	60.25	-78.65	-108.3	-125.7	-137.9	-148.2	-157.4	-166.1	-174.4	-182.5	-190.4	-198.2
50	30800	4434	1474	682.3	355.0	25.97	-46.75	-79.82	-103.4	-121.3	-136.1	-148.1	-160.3	-170.9	-180.9	-190.5
60	63850	9204	3116	1494	864.5	176.0	55.03	-0.3403	-35.84	-61.98	-82.73	-100.0	-115.0	-128.4	-140.6	-151.9
70	118900	16990	5733	2752	1603	372.4	179.7	101.8	85.83	21.88	-3.959	-25.11	-43.04	-58.66	-72.56	-85.20
80	205300	29020	9713	4631	2621	628.4	329.4	221.2	161.7	122.6	92.92	69.19	49.48	32.64	17.96	4.873
90	334700	46870	15550	7353	41.48	963.6	510.0	358.5	283.1	236.2	202.9	177.6	157.3	140.3	125.9	113.4
	VALUES OF K_1 AT $\frac{1}{4}$ POINT															
10	-397.1	-111.1	-736.3	-60.68	-54.90	-48.27	-48.46	-50.21	-52.22	-54.24	-56.53	-58.69	-61.06	-63.27	-65.70	-67.93
20	-63.74	-237.9	-187.2	-152.5	-131.9	-100.9	-97.16	-98.38	-101.2	-104.7	-108.5	-112.5	-116.7	-120.9	-125.2	-129.5
30	2097	96.48	-91.04	-125.7	-132.3	-127.5	-127.6	-131.0	-135.6	-140.8	-146.3	-151.	-157.9	-163.8	-169.8	-175.8
40	7520	968.4	243.4	-54.70	-16.84	-98.08	-118.9	-132.0	-142.4	-151.5	-160.0	-168.1	-175.9	-183.6	-191.2	-198.7
50	18580	2645	856.0	378.0	177.1	-21.49	-71.40	-96.91	-116.1	-131.2	-144.1	-155.4	-165.8	-175.6	-184.9	-193.8
60	38480	5553	1873	890.2	508.4	8562	5.504	-34.72	-62.22	-83.35	-100.6	-115.4	-128.4	-140.2	-151.0	-161.2
70	71380	10250	3470	1669	972.8	220.5	97.12	43.48	9.271	-16.26	-36.77	-53.96	-68.83	-81.99	-93.85	-104.8
80	122500	17440	5869	2814	1580	390.0	202.9	131.9	90.73	62.34	40.04	21.75	6.240	-7.236	-19.14	-29.89
90	198600	27990	9346	4449	2575	605.5	326.0	229.8	180.2	148.2	125.9	106.4	91.28	78.55	67.62	57.99
	VALUES OF K_1 AT $\frac{1}{2}$ POINT															
10	-307.8	-101.6	-69.69	-58.65	-53.64	-48.03	-48.72	-50.11	-52.20	-54.14	-56.46	-58.57	-60.96	-63.14	-65.56	-67.78
20	-707.7	-296.9	-197.0	-154.2	-132.1	-100.9	-97.26	-98.39	-101.1	-104.4	-108.2	-112.1	-115.6	-120.3	-124.5	-128.7
30	-1569	-437.4	-269.1	-209.7	-179.8	-138.9	-133.4	-134.6	-138.0	-142.3	-147.3	-152.5	-157.9	-163.4	-169.0	-174.7
40	-3879	-741.6	-358.9	-246.7	-199.6	-147.1	-144.9	-149.0	-154.6	-160.6	-167.0	-173.4	-180.0	-186.6	-193.2	-199.9
50	-8833	-1406	-559.9	-327.9	-244.5	-140.1	-136.9	-142.3	-150.2	-158.2	-166.0	-173.6	-181.0	-188.4	-195.4	-202.8
60	-18080	-2652	-948.0	-494.5	-319.0	-136.7	-121.2	-124.8	-132.4	-140.8	-149.1	-157.1	-164.9	-172.4	-179.7	-186.8
70	-33680	-4778	-1616	-788.8	-473.0	-147.0	-100.3	-106.8	-111.0	-117.4	-113.1	-131.6	-138.5	-145.2	-151.7	-158.0
80	-59250	-8181	-2689	-1265	-635.8	-176.7	-109.5	-94.62	-92.91	-95.25	-99.43	-104.2	-109.1	-114.0	-118.8	-123.5
90	-98200	-13390	-4330	-1997	-1121	-231.8	-120.8	-90.98	-81.23	-78.49	-78.68	-80.16	-82.23	-84.54	-86.92	-89.33
	VALUES OF K_1 AT $\frac{3}{4}$ POINT															
10	-206.9	-84.00	-62.65	-55.06	-51.53	-47.61	-48.33	-49.95	-51.87	-53.98	-56.14	-58.38	-60.64	-62.92	-65.22	-67.54
20	-1015	-299.6	-185.8	-145.5	-126.3	-100.2	-97.13	-98.26	-100.8	-104.0	-107.6	-111.4	-115.3	-119.3	-123.4	-127.5
30	-3719	-760.6	-380.9	-265.3	-214.9	-150.2	-140.2	-139.1	-141.0	-144.3	-148.4	-152.8	-157.6	-162.6	-167.7	-172.9
40	-10550	-1815	-767.8	-468.7	-345.1	-198.6	-176.0	-170.7	-170.7	-173.0	-176.5	-180.7	-185.4	-190.4	-195.7	-201.1
50	-24640	-3888	-1496	-833.0	-569.4	-260.6	-213.3	-198.9	-194.5	-194.0	-195.7	-198.4	-201.9	-206.0	-210.4	-215.0
60	-49750	-7525	-2740	-1440	-925.3	-351.0	-262.0	-232.9	-220.7	-215.2	-213.2	-213.1	-214.3	-216.1	-218.7	-221.6
70	-91360	-13420	-4718	-2386	-1476	-480.5	-332.6	-279.7	-256.5	-244.0	-254.5	-232.6	-261.8	-229.2	-269.0	-229.4
80	-155900	-22430	-7703	-3795	-2288	-659.7	-422.3	-343.2	-306.2	-385.1	-271.8	-262.8	-256.4	-251.7	-248.3	-245.9
90	-251500	-35840	-12030	-5815	-3434	-901.4	-542.8	-425.8	-375.2	-340.6	-320.7	-306.7	-296.2	-288.1	-281.6	-276.3
	VALUES OF K_1 AT ABUTMENT															
10	-37.55	-38.92	-39.50	-39.99	-40.46	-42.73	-45.02	-47.34	-49.67	-52.00	-54.34	-56.69	-59.03	-61.38	-63.73	-66.08
20	-72.53	-73.50	-74.66	-75.71	-76.64	-80.59	-84.41	-88.32	-92.31	-96.36	-100.5	-104.6	-108.8	-112.9	-117.1	-121.3
30	-108.8	-108.9	-109.3	-109.8	-110.4	-113.7	-117.3	-121.4	-125.7	-130.3	-135.1	-139.9	-144.9	-149.9	-155.0	-160.1
40	-145.6	-145.8	-146.0	-146.2	-146.4	-147.3	-148.6	-150.7	-153.6	-157.0	-160.9	-165.0	-169.4	-174.0	-178.8	-183.6
50	-182.6	-183.1	-183.6	-184.1	-184.5	-184.6	-183.6	-182.8	-182.8	-183.6	-185.1	-187.1	-189.6	-192.4	-195.6	-198.9
60	-219.7	-220.7	-221.7	-222.6	-223.3	-225.0	-223.8	-221.3	-218.7	-216.5	-215.0	-214.1	-213.8	-214.0	-214.7	-215.7
70	-256.9	-258.5	-260.0	-261.4	-262.7	-267.0	-267.6	-265.6	-262.2	-258.4	-254.7	-251.3	-248.4	-246.0	-244.0	-242.5
80	-294.3	-296.4	-298.5	-300.4	-302.3	-309.7	-313.5	-313.9	-311.9	-308.4	-304.3	-299.9	-295.6	-291.6	-287.8	-284.3
90	-331.9	-334.5	-337.1	-339.7	-342.2	-352.9	-360.4	-364.3	-365.4	-364.4	-361.9	-358.5	-354.7	-350.6	-346.5	-342.5

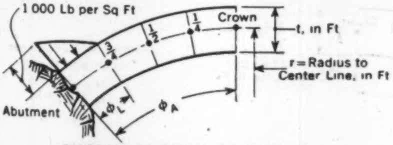


DIAGRAM OF RADIAL LOAD NO 2

Radial Deflection, in Ft

$$\Delta_t = K_1 \frac{r}{E_C}$$

E_C = Modulus of Elasticity of Concrete in Direct Stress, Lb per Sq Ft

TABLE 5(*a*).—FORCES AND MOMENTS FOR RADIAL LOAD No. 3

ϕ_A	VALUES OF t/r															
	.025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
VALUES OF h AT CROWN																
10	22.85	11.32	8.388	7.387	7.014	7.146	7.741	8.330	8.893	9.429	9.951	10.46	10.96	11.46	11.95	12.43
20	57.27	48.56	40.40	34.91	31.51	27.19	28.30	30.26	32.37	34.49	36.57	38.62	40.63	42.62	44.58	46.59
30	61.73	62.89	62.07	60.41	58.64	54.73	56.69	60.55	64.96	69.50	74.04	78.53	82.97	87.34	89.84	95.95
40	62.30	65.52	67.91	69.69	71.07	76.56	83.16	90.82	98.82	106.9	114.8	122.6	130.3	137.9	145.4	152.8
50	62.69	66.14	69.28	72.17	74.89	87.56	100.3	113.3	126.2	138.8	151.1	163.2	174.9	186.5	197.8	208.9
60	63.31	66.64	69.84	72.96	76.05	91.69	108.4	125.9	143.6	161.0	178.1	194.7	211.0	226.9	242.5	257.9
70	64.21	67.36	70.46	73.56	76.66	93.02	111.3	131.3	152.2	173.5	194.7	215.6	236.3	256.5	276.4	296.0
80	65.38	68.37	71.35	74.36	77.38	93.54	112.0	132.6	155.3	178.9	202.9	227.2	251.3	275.4	299.1	322.8
90	66.81	69.68	72.55	75.45	78.38	94.09	112.2	132.8	155.6	180.0	205.6	231.9	258.6	285.3	312.1	338.9
VALUES OF m AT CROWN																
10	.0958	0.216	0.286	0.342	0.391	0.586	0.739	0.870	0.986	1.093	1.193	1.289	1.380	1.469	1.555	1.639
20	-0.450	-0.182	.0905	0.313	0.491	1.071	1.472	1.815	2.127	2.420	2.699	2.967	3.228	3.481	3.729	4.215
30	-1.249	-1.195	-1.044	-0.846	-0.637	0.201	0.715	1.084	1.390	1.661	1.913	2.153	2.384	2.608	2.828	3.043
40	-2.256	2.369	-2.421	-2.426	-2.401	-2.170	-2.090	-2.174	-2.350	-2.575	-2.828	-3.096	-3.373	-3.655	-3.941	-4.229
50	-3.512	-3.743	-3.946	-4.129	-4.300	-5.146	-6.178	-7.402	-8.777	-10.16	-11.61	-13.09	-14.57	-16.06	-17.55	-19.05
60	-5.030	-5.360	-5.680	-6.000	-6.326	-8.180	-10.54	-13.35	-16.44	-19.71	-23.08	-26.51	-29.98	-33.47	-36.99	-40.50
70	-6.820	-7.239	-7.661	-8.095	-8.546	-11.22	-14.77	-19.15	-24.18	-29.64	-35.40	-41.36	-47.45	-53.64	-59.90	-66.21
80	-8.884	-9.390	-9.908	-10.44	-11.00	-14.34	-18.84	-24.56	-31.33	-38.94	-47.18	-55.87	-64.91	-74.19	-83.68	-93.31
90	-11.22	-11.82	-12.43	-13.06	-13.72	-17.62	-22.87	-29.62	-37.81	-47.25	-57.71	-68.99	-80.91	-93.34	-106.2	-1.93
VALUES OF h AT ABUTMENT																
10	23.78	12.45	9.577	8.607	8.256	8.465	9.130	9.790	10.42	11.03	11.62	12.20	2.78	13.34	13.91	14.46
20	58.95	50.83	43.23	38.13	35.00	31.26	32.82	34.77	37.07	39.38	41.65	43.89	46.10	48.28	50.45	52.85
30	64.99	66.13	65.56	64.27	62.88	60.21	62.61	66.67	71.20	75.85	80.49	85.09	89.64	94.14	97.02	103.0
40	68.16	70.88	72.96	74.58	75.89	81.36	87.67	94.80	102.2	109.6	117.0	124.2	131.4	138.5	145.4	152.4
50	72.12	74.73	77.14	79.39	81.54	91.64	101.8	112.1	122.4	132.4	142.3	152.0	161.6	170.9	180.2	189.3
60	77.29	79.52	81.68	83.81	85.91	96.55	107.7	119.3	130.9	142.5	153.8	165.0	175.9	186.7	197.3	207.8
70	83.77	85.61	87.43	89.25	91.08	100.5	110.6	121.2	132.2	143.3	154.6	165.3	176.2	186.9	197.6	208.1
80	91.62	93.13	94.64	96.15	97.67	105.4	113.6	122.2	131.0	140.1	149.2	158.4	167.5	176.6	185.7	94.8
90	100.9	102.2	103.4	104.7	105.9	112.1	118.4	124.6	130.8	137.1	143.3	149.5	155.8	162.0	168.2	174.4
VALUES OF m AT ABUTMENT																
10	-0.842	-0.913	-0.903	-0.878	-0.851	-0.733	-0.650	-0.589	-0.544	-0.508	-0.479	-0.456	-0.436	-0.419	-0.405	-0.392
20	-2.128	-2.449	-2.732	-2.904	-2.995	-2.992	-2.841	-2.696	-2.574	-2.470	-2.383	-2.308	-2.243	-2.186	-2.137	-1.946
30	-4.505	-4.433	-4.540	-4.707	-4.875	-5.274	-5.209	-5.033	-4.849	-4.680	-4.531	-4.402	-4.288	-4.189	-4.095	-4.024
40	-8.117	-7.729	-7.475	-7.317	-7.219	-6.966	-6.604	-6.158	-5.722	-5.327	-4.981	-4.679	-4.416	-4.187	-3.985	-3.806
50	-12.94	-12.33	-11.81	-11.35	-10.94	-9.228	-7.677	-6.230	-4.960	-3.792	-2.809	-1.940	-1.187	-0.526	0.058	0.577
60	-19.01	-18.24	-17.52	-16.84	-16.19	-13.04	-9.855	-6.733	-3.820	-1.186	1.164	3.250	5.103	6.755	8.230	9.562
70	-26.38	-25.49	-24.63	-23.79	-22.96	-18.69	-14.00	-9.048	-4.123	0.575	5.201	8.955	12.61	15.95	18.99	21.76
80	-35.12	-34.15	-33.19	-32.24	-31.29	-26.22	-20.39	-13.88	-7.021	-0.136	6.558	12.93	18.93	24.53	29.74	34.38
90	-45.34	-44.32	-43.30	-42.28	-41.25	-35.68	-29.09	-21.46	-13.07	-4.271	4.622	13.39	21.89	30.02	37.77	45.10
VALUES OF v AT ABUTMENT																
10	-40.18	-42.73	-43.78	-44.50	-45.11	-47.82	-50.44	-53.06	-55.69	-58.32	-60.95	-63.59	-66.25	-68.87	-71.51	-74.15
20	-68.55	-72.61	-76.49	-79.46	-81.71	-88.62	-93.69	-98.46	-103.2	-107.9	-112.6	-117.4	-122.1	-126.9	-131.6	-138.4
30	-100.9	-102.0	-104.0	-106.5	-109.0	-119.1	-126.2	-132.4	-138.3	-144.2	-150.1	-156.0	-161.9	-167.8	-174.7	-179.8
40	-134.9	-135.0	-135.6	-136.8	-137.9	-145.2	-151.7	-157.6	-163.2	-168.9	-174.6	-180.3	-186.2	-192.1	-198.1	-204.1
50	-169.4	-169.4	-169.7	-170.2	-170.8	-174.5	-178.2	-181.6	-185.2	-188.9	-192.9	-197.1	-201.5	-206.1	-210.8	-215.7
60	-204.2	-204.6	-205.0	-205.5	-206.0	-208.4	-210.0	-210.8	-211.5	-212.4	-213.6	-215.2	-217.1	-219.3	-221.7	-224.4
70	-239.4	-240.2	-240.9	-241.7	-242.5	-245.6	-246.9	-246.7	-245.5	-244.0	-242.6	-241.4	-240.6	-240.0	-239.8	-238.9
80	-274.9	-276.2	-277.4	-278.6	-279.9	-284.9	-287.5	-288.1	-286.9	-284.6	-281.9	-279.0	-276.1	-273.4	-270.9	-268.7
90	-310.8	-312.6	-314.4	-316.1	-317.8	-325.4	-330.7	-333.4	-333.9	-332.7	-330.4	-327.5	-324.1	-320.8	-317.2	-313.7

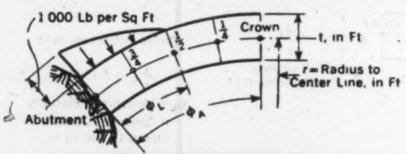


DIAGRAM OF RADIAL LOAD NO. 3

Thrust, Moment, Shear, in Lb, and Ft-Lb

$H = hr$

$M = mr^2$

$V = vr$

TABLE 5(*b*).—Radial Deflections for Radial Load No. 3

VALUES OF K₁ AT CROWN

ϕ_A	t/r .025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
10	-1614	-456.5	-246.3	-177.3	-146.6	-109.0	-104.9	-106.1	-108.9	-112.5	-116.5	-120.6	-125.0	-129.4	-133.9	-138.5
20	-374.3	-1337	-904.9	-642.8	-494.8	-272.7	-233.4	-224.1	-220.8	-227.6	-233.5	-239.8	-247.1	-254.8	-262.8	-270.8
30	14770	489.8	-600.1	-701.0	-654.4	-423.9	-351.8	-329.2	-323.7	-325.7	-331.5	-339.4	-348.6	-358.6	-369.0	-380.3
40	55080	6197	1271	132.9	-228.4	-419.9	-390.0	-376.4	-374.9	-379.7	-387.9	-398.3	-409.9	-422.4	-435.5	-449.0
50	139900	17970	5168	1974	824.4	-211.5	-305.3	-332.9	-351.8	-367.3	-383.4	-399.4	-415.5	-431.8	-448.0	-464.3
60	295900	39250	12080	5194	2652	186.2	-104.8	-198.2	-248.3	-283.5	-312.1	-337.0	-359.8	-381.1	-401.4	-421.0
70	558800	74640	23430	10400	5564	784.7	197.6	13.14	-78.28	-136.9	-180.6	-216.0	-237.5	-273.3	-298.0	-320.9
80	974700	130200	41070	18400	9984	1634	606.0	293.4	148.1	61.25	0.733	-45.74	-83.75	-116.2	-144.7	-170.4
90	1603000	213700	67390	29680	16470	2814	1143	647.4	428.3	305.2	224.5	165.9	120.2	88.77	51.10	23.47

VALUES OF K₁ AT ¼ POINT

ϕ_A	t/r .025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
10	-1543	-437.1	-238.7	-173.5	-144.5	-108.6	-104.7	-105.9	-108.8	-112.4	-116.4	-120.6	-124.7	-129.3	-133.8	-138.4
20	-1615	-1402	-897.1	-630.4	-484.8	-270.1	-232.2	-223.5	-223.5	-227.2	-232.9	-239.3	-246.6	-254.3	-262.2	-270.3
30	6935	-465.0	-855.9	-793.7	-693.9	-425.9	-352.4	-329.6	-323.9	-325.7	-331.3	-338.9	-348.0	-357.8	-368.3	-379.1
40	29620	2811	225.1	-321.5	-469.4	-455.1	-407.0	-384.6	-383.5	-383.5	-394.1	-400.1	-414.7	-423.0	-439.6	-448.6
50	76930	9496	2483	765.7	165.4	-326.9	-354.5	-362.3	-371.9	-382.6	-395.4	-408.9	-423.2	-437.8	-452.8	-468.0
60	163300	21450	6428	2636	1247	-70.18	-215.4	-266.9	-297.1	-321.2	-342.5	-362.3	-381.2	-399.4	-417.3	-434.7
70	307800	41110	12810	5605	2934	307.1	-9.847	-116.3	-171.7	-210.3	-241.2	-267.6	-291.3	-313.1	-333.5	-352.9
80	534600	71680	22620	10100	5449	826.1	255.2	77.92	-8.391	-63.07	-103.4	-136.0	-163.7	-188.2	-210.4	-230.8
90	874500	117200	37090	16670	9083	1528	328.0	314.4	186.9	112.6	52.71	23.25	-7.828	-34.08	-56.91	-77.33

VALUES OF K₁ AT ½ POINT

ϕ_A	t/r .025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
10	- 1299	- 376.8	- 215.5	-161.9	-137.7	-107.3	-104.2	-105.6	-108.5	-112.1	-116.1	-120.3	-124.6	-129.0	-133.5	-138.1
20	- 4056	- 1461	- 840.4	-581.0	-449.3	-261.8	-229.1	-221.6	-222.0	-225.9	-231.4	-237.9	-245.1	-252.7	-263.5	-268.5
30	- 9509	- 2430	- 1360	-963.9	-758.8	-425.6	-352.3	-329.7	-323.8	-325.2	-330.3	-337.4	-345.9	-355.2	-365.2	-375.6
40	- 23970	- 4344	- 1989	-1284	-972.2	-532.1	-436.5	-404.3	-394.0	-393.3	-397.6	-404.9	-413.9	-424.1	-435.2	-446.8
50	- 55040	- 8399	- 3234	-1828	-1263	-590.8	-534.9	-435.7	-423.9	-422.4	-426.7	-434.0	-443.3	-453.9	-465.3	-477.4
60	-113100	-15960	- 5561	-2838	-1791	-655.9	-487.8	-438.0	-421.9	-419.0	-422.4	-429.3	-438.2	-448.4	-459.4	-471.1
70	-212200	-28880	- 9531	-4569	-2702	-771.7	-510.6	-434.5	-407.4	-398.8	-398.8	-403.1	-410.0	-418.4	-427.8	-437.8
80	-371000	-49630	-15880	-7342	-3145	-972.4	-562.7	-443.0	-396.8	-377.5	-370.5	-369.7	-372.4	-377.1	-383.2	-390.2
90	-613700	-81020	-25560	-11580	-6411	-1293	-660.1	-476.1	-401.7	-366.6	-349.0	-340.4	-336.9	-336.3	-337.6	-340.3

VALUES OF K₁ AT ¾ POINT

ϕ_A	t/r .025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
10	- 779.9	- 261.6	- 169.4	- 137.3	- 122.3	-103.0	-101.8	-103.8	-107.0	-110.8	-114.9	-119.1	-123.5	-127.9	-132.4	-137.0
20	- 3978	- 1094	- 616.3	- 441.7	- 357.0	-237.7	-217.1	-213.4	-215.5	-220.1	-226.0	-232.7	-239.9	-247.5	-255.2	-263.2
30	- 14380	- 2770	- 1310	- 862.0	- 661.7	-386.2	-332.3	-316.4	-313.1	-315.5	-320.8	-327.9	-336.1	-345.0	-354.6	-364.4
40	- 41310	- 6668	- 2668	- 1549	-996.0	-535.6	-436.4	-403.7	-392.3	-390.1	-392.7	-398.3	-405.6	-413.4	-423.6	-433.7
50	- 97470	- 14550	- 5303	- 2808	- 1825	-718.8	-541.3	-482.3	-458.0	-448.0	-445.6	-447.2	-451.7	-458.0	-465.6	-474.1
60	- 199900	- 28670	- 9932	- 4975	- 3058	-984.8	-674.1	-571.2	-524.0	-503.1	-491.9	-487.1	-486.3	-488.2	-492.2	-497.2
70	- 370800	- 51940	-17460	- 8449	- 5006	- 1378	-859.1	-689.8	-612.7	-571.6	-546.5	-533.4	-524.9	-520.3	-518.4	-518.5
80	- 638300	- 88030	-29020	-13730	- 7946	- 1942	- 1117	-852.6	-732.3	-666.3	-625.9	-599.3	-580.3	-568.4	-559.4	-553.2
90	-1038000	-141600	-46050	-21460	-12210	- 2730	- 1467	- 1071	-893.1	-798.5	-734.9	-693.9	-664.7	-642.8	-626.1	-613.0

VALUES OF K₁ AT ABUTMENT

ϕ_A	t/r .025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
10	-71.73	-76.27	-78.15	-79.44	-80.53	-85.35	-90.03	-94.71	-99.40	-104.1	-108.8	-113.5	-118.2	-122.9	-127.6	-132.4
20	-122.4	-129.6	-136.5	-141.8	-145.8	-158.2	-167.2	-175.7	-184.2	-192.6	-201.0	-209.5	-218.0	-226.5	-235.0	-243.5
30	-180.1	-182.0	-185.6	-190.0	-194.5	-212.5	-225.3	-236.4	-247.0	-257.4	-267.9	-278.4	-289.0	-299.6	-310.2	-320.9
40	-240.8	-240.9	-242.0	-243.9	-264.0	-259.1	-270.8	-281.3	-291.4	-301.4	-311.6	-321.9	-332.3	-342.9	-353.6	-364.4
50	-302.4	-302.4	-302.9	-303.8	-304.8	-311.5	-318.0	-324.8	-330.5	-337.2	-344.4	-351.8	-359.7	-367.9	-376.4	-385.1
60	-364.6	-365.1	-365.9	-366.8	-367.7	-372.1	-374.8	-376.3	-377.5	-379.1	-381.3	-384.1	-387.4	-391.4	-395.8	-400.6
70	-427.4	-428.7	-430.1	-431.5	-432.9	-438.5	-440.8	-440.3	-438.2	-435.5	-433.1	-431.0	-429.4	-428.5	-428.1	-428.2
80	-490.7	-492.9	-495.2	-497.4	-499.6	-506.5	-513.4	-514.2	-512.1	-508.1	-503.2	-498.0	-492.8	-488.0	-483.6	-479.6
90	-544.7	-557.9	-561.1	-564.3	-567.4	-580.9	-590.3	-595.1	-596.0	-593.9	-589.8	-584.5	-578.6	-572.4	-566.1	-560.0

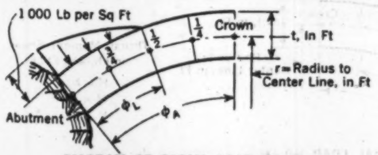


DIAGRAM OF RADIAL LOAD NO. 3

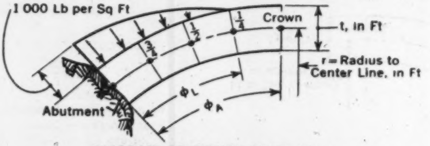
Radial Deflection, in Ft

$$\Delta_r = K_1 \frac{r}{E_c}$$

E_c = Modulus of Elasticity of Concrete in Direct Stress, Lb per Sq Ft

TABLE 6(*a*).—Forces and Moments for Radial Load No. 4

ϕ_A	VALUES OF t/r															
	.025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
VALUES OF h AT CROWN																
10	56.39	24.62	16.38	13.33	11.97	10.83	11.31	11.97	12.66	13.34	14.01	14.68	15.35	16.01	16.67	17.32
20	150.7	119.8	93.43	75.86	64.69	46.42	44.63	45.89	48.01	50.42	52.95	55.52	58.10	60.69	63.27	65.98
30	165.9	161.8	153.0	142.6	132.7	103.8	96.81	97.45	100.9	105.5	110.6	115.9	121.4	127.0	132.5	138.1
40	169.0	172.0	172.6	171.5	169.4	157.0	152.7	155.2	161.1	168.8	177.3	186.3	195.6	205.0	214.4	223.9
50	170.9	175.5	179.1	181.9	184.0	189.8	195.7	204.6	215.7	228.3	241.6	255.5	269.6	283.7	297.9	312.1
60	173.0	178.0	182.5	186.6	190.4	206.7	222.2	238.8	256.8	275.6	296.0	314.6	334.3	354.0	373.6	393.2
70	175.6	180.6	185.4	189.9	194.4	215.6	237.0	259.7	283.7	308.6	333.9	359.5	385.1	410.6	436.1	461.6
80	178.7	183.6	188.4	193.2	198.2	221.2	245.7	272.1	300.2	329.6	359.9	390.6	421.6	452.6	483.5	514.3
90	182.4	187.2	192.0	196.8	201.6	225.8	251.7	280.0	310.4	342.8	376.4	411.0	446.2	481.7	517.4	553.1
VALUES OF m AT CROWN																
10	0.357	0.654	0.817	0.943	1.052	1.490	1.834	2.129	2.394	2.637	2.866	3.084	3.295	3.499	3.698	3.893
20	-0.781	.0394	0.818	1.433	1.916	3.452	4.508	5.411	6.236	7.011	7.750	8.463	9.156	9.832	10.49	11.49
30	-2.499	-2.112	-1.469	-0.714	0.052	3.080	5.047	6.556	7.856	9.044	10.17	11.24	12.29	13.31	14.31	15.30
40	-4.642	-4.611	-4.383	-4.001	-3.513	-0.679	1.641	3.362	4.716	5.855	6.865	7.793	8.666	9.502	10.31	11.10
50	-7.313	-7.521	-7.608	-7.590	-7.484	-6.274	-4.967	-4.059	-3.532	-3.279	-3.200	-3.273	-3.420	-3.627	-3.877	-4.159
60	-10.56	-10.95	-11.27	-11.53	-11.74	-12.37	-12.94	-13.87	-15.19	-16.85	-18.75	-20.83	-23.04	-25.34	-27.72	-30.16
70	-14.40	-14.95	-15.47	-15.95	-16.42	-18.66	-21.22	-24.40	-28.20	-32.52	-37.25	-42.28	-47.54	-52.97	-58.54	-64.21
80	-18.85	-19.56	-20.25	-20.93	-21.70	-25.20	-29.54	-34.90	-41.27	-48.53	-56.50	-65.02	-73.98	-83.29	-92.86	-102.6
90	-23.92	-24.79	-25.65	-26.51	-27.39	-32.14	-37.99	-45.24	-53.95	-63.98	-75.15	-87.26	-100.1	-113.6	-127.6	-142.0
VALUES OF h AT ABUTMENT																
10	58.42	27.17	19.09	16.12	14.82	13.88	14.53	15.36	16.21	17.06	17.90	18.74	19.57	20.40	21.23	22.05
20	153.2	124.2	99.60	83.24	72.88	56.43	55.45	57.35	60.05	63.03	66.12	69.25	72.39	75.53	78.66	81.92
30	169.5	166.3	158.9	150.2	142.0	118.6	114.1	116.3	120.8	126.4	132.4	138.7	145.0	151.4	157.8	164.2
40	175.1	177.9	179.0	178.7	177.6	171.0	170.5	175.2	182.6	191.2	200.6	210.3	220.2	230.2	240.3	250.4
50	180.6	184.5	187.6	190.3	192.5	200.6	208.8	218.9	230.4	242.8	255.8	269.1	282.5	295.9	309.4	322.9
60	187.4	191.2	194.7	198.0	201.1	225.5	229.5	244.0	259.2	274.9	291.3	306.8	322.9	339.0	355.0	371.1
70	195.9	199.3	202.6	205.8	209.0	224.7	240.4	256.6	273.1	290.0	307.5	324.2	341.3	358.5	375.6	392.6
80	206.2	209.2	212.2	215.2	218.2	233.0	248.1	263.5	279.2	295.1	311.2	327.3	343.5	359.7	375.9	392.1
90	218.5	221.2	223.9	226.6	229.3	242.8	256.2	269.7	283.2	296.7	310.2	323.7	337.2	350.6	364.1	377.6
VALUES OF m AT ABUTMENT																
10	-1.675	-1.897	-1.895	-1.851	-1.797	-1.556	-1.382	-1.256	-1.169	-1.084	-1.023	-0.972	-0.950	-0.895	-0.864	-0.837
20	-3.218	-4.405	-5.358	-5.946	-6.279	-6.555	-6.319	-6.052	-5.811	-5.602	-5.421	-5.265	-5.128	-5.008	-4.901	-4.452
30	-6.098	-6.578	-7.438	-8.392	-9.275	-11.71	-12.27	-12.27	-12.10	-11.90	-11.68	-11.48	-11.30	-11.13	-10.98	-10.84
40	-10.73	-10.57	-10.77	-11.21	-11.78	-14.65	-16.15	-16.68	-16.75	-16.64	-16.44	-16.22	-16.00	-15.79	-15.58	-15.40
50	-17.02	-16.45	-16.12	-16.00	-16.02	-17.09	-18.03	-18.33	-18.18	-17.80	-17.34	-16.82	-16.32	-15.84	-15.38	-14.99
60	-24.99	-24.13	-23.44	-22.89	-22.45	-21.18	-20.24	-19.05	-17.63	-16.09	-14.05	-13.05	-11.64	-10.33	-9.111	-7.976
70	-34.73	-33.67	-32.72	-31.86	-31.08	-27.77	-24.61	-21.21	-17.62	-13.98	-9.978	-7.022	-3.821	-0.830	1.954	4.541
80	-46.37	-45.16	-44.03	-42.96	-41.78	-37.06	-31.95	-26.31	-20.25	-14.01	-7.785	-1.740	4.055	9.552	14.74	19.81
90	-60.06	-58.76	-57.50	-56.29	-55.10	-49.13	-42.53	-35.03	-26.74	-17.94	-8.920	.0813	8.903	17.44	25.63	33.44
VALUES OF v AT ABUTMENT																
10	-56.38	-62.72	-64.96	-66.31	-67.36	-71.64	-75.65	-79.62	-83.58	-87.55	-91.52	-95.48	-99.45	-103.4	-107.4	-111.4
20	-80.23	-92.44	-103.1	-110.7	-116.2	-130.6	-139.3	-147.0	-154.4	-161.7	-169.0	-176.2	-183.5	-190.7	-198.0	-205.2
30	-113.3	-117.8	-124.6	-132.2	-139.6	-166.2	-181.8	-193.6	-204.0	-213.8	-223.4	-232.8	-242.2	-251.5	-260.8	-270.2
40	-150.4	-151.7	-154.6	-158.4	-163.0	-186.9	-205.7	-220.1	-232.3	-243.4	-253.9	-264.0	-274.1	-284.0	-294.0	-303.9
50	-188.8	-189.2	-190.3	-192.2	-194.5	-209.8	-225.0	-237.9	-249.1	-259.2	-268.8	-277.9	-286.8	-295.7	-304.6	-313.4
60	-227.8	-228.1	-228.8	-230.0	-231.3	-240.5	-250.4	-259.3	-267.1	-274.0	-279.7	-286.9	-293.2	-299.4	-305.7	-312.0
70	-267.3	-268.0	-268.8	-269.8	-271.0	-277.8	-284.3	-289.6	-293.8	-297.1	-300.0	-302.7	-305.3	-308.0	-310.8	-313.7
80	-307.5	-308.6	-309.8	-311.1	-312.2	-319.3	-325.0	-328.9	-331.0	-331.9	-331.9	-331.5	-330.9	-330.2	-329.6	-329.0
90	-348.2	-349.9	-351.6	-353.4	-355.2	-363.7	-370.5	-375.0	-377.3	-377.7	-376.8	-375.0	-372.5	-369.8	-366.9	-363.9

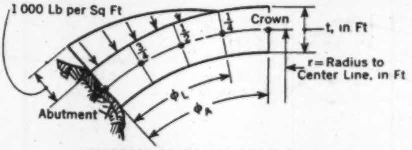


Thrust, Moment, Shear,
in Lb, and Ft-Lb
H = hr
$M = mr^2$
V = vr

DIAGRAM OF RADIAL LOAD NO. 4

TABLE 6(*b*).—RADIAL DEFLECTIONS FOR RADIAL LOAD NO. 4

ϕ_A	VALUES OF t/r															
	025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
VALUES OF K_1 AT CROWN																
10	-4316	-1078	-527.7	-352.7	-276.4	-182.1	-168.6	-167.4	-170.2	-174.6	-179.9	-185.7	-191.9	-198.3	-204.8	-211.5
20	-4865	-3974	-2383	-1572	-1141	-521.2	-410.2	-377.6	-370.3	-368.5	-373.5	-381.1	-390.4	-400.7	-411.8	-423.0
30	21420	-1507	-2514	-2216	-1850	-937.7	-690.0	-603.1	-569.1	-557.5	-557.0	-562.6	-572.0	-583.8	-597.3	-612.0
40	93330	-8255	441.3	-1083	-1418	-1131	-881.4	-770.5	-720.9	-700.5	-695.3	-698.9	-707.9	-720.6	-735.6	-752.4
50	245700	28910	7088	1937	221.0	-944.9	-875.5	-807.1	-771.1	-756.5	-754.9	-761.1	-772.6	-787.6	-805.0	-824.2
60	527400	66620	19120	7422	3263	-408.4	-657.9	-690.9	-698.3	-706.4	-712.3	-734.0	-752.2	-772.4	-794.0	-816.7
70	1003000	129800	39050	16420	-8212	522.9	-249.1	-436.0	-510.6	-554.6	-588.5	-618.5	-647.0	-674.7	-711.9	-729.1
80	1759000	229300	70240	30390	16020	1882	351.8	-55.27	-222.4	-314.2	-376.0	-424.7	-465.7	-502.2	-535.9	-567.8
90	2902000	379400	117000	51200	27090	3815	1176	456.2	161.6	6.154	-91.88	-162.5	-217.8	-264.1	-304.4	-340.8
VALUES OF K_1 AT $\frac{1}{4}$ POINT																
10	-4053	-1018	-505.9	-342.0	-270.0	-181.0	-168.2	-167.2	-170.0	-174.4	-179.8	-185.6	-191.7	-198.1	-204.6	-211.3
20	-6963	-3954	-2298	-1510	-1098	-511.8	-406.5	-375.6	-366.9	-367.4	-372.5	-380.2	-389.4	-399.7	-410.7	-422.0
30	6153	-3150	-2866	-2298	-1857	-922.7	-686.1	-599.1	-566.4	-555.3	-554.9	-560.6	-569.9	-581.6	-594.9	-609.4
40	42350	1833	-1409	-1820	-1766	-1155	-885.7	-772.1	-721.8	-700.7	-695.0	-698.1	-706.6	-718.6	-733.1	-749.3
50	118400	12370	2043	-234.0	-906.9	-1095	-926.6	-834.0	-788.6	-768.9	-764.1	-767.8	-777.4	-790.7	-806.6	-824.6
60	258100	31380	8222	2637	713.5	-797.4	-806.4	-773.0	-753.2	-746.9	-744.4	-759.4	-772.9	-789.3	-811.4	-827.7
70	492600	62910	18330	7275	3308	-263.1	-557.7	-610.7	-629.4	-644.1	-660.0	-677.9	-697.2	-717.9	-747.9	-761.8
80	862100	112200	33960	14370	6678	494.5	-196.9	-367.9	-437.0	-447.7	-509.0	-536.6	-562.5	-587.5	-612.0	-636.3
90	1418000	185800	57170	24810	12950	1544	283.2	-51.13	-186.9	-260.6	-314.0	-348.5	-381.0	-410.0	-436.8	-462.2
VALUES OF K_1 AT $\frac{1}{2}$ POINT																
10	-3180	-831.0	-435.4	-306.7	-248.9	-176.3	-165.8	-165.6	-168.7	-173.3	-178.7	-184.6	-190.8	-197.2	-203.8	-210.4
20	-9897	-3548	-1954	-1287	-950.7	-477.3	-391.2	-366.2	-360.0	-361.6	-367.3	-375.3	-384.7	-395.0	-406.0	-417.3
30	-21290	-5799	-3265	-2272	-1742	-853.8	-649.2	-578.6	-551.6	-543.0	-543.9	-550.0	-559.5	-571.1	-584.2	-598.3
40	-51040	-9779	-4647	-3039	-2293	-1150	-868.3	-757.2	-709.7	-689.8	-684.1	-686.7	-694.4	-705.4	-718.8	-733.9
50	-114900	-18070	-7212	-4190	-2939	-1340	-1002	-870.6	-810.5	-782.7	-772.2	-771.4	-777.0	-786.8	-799.5	-814.3
60	-234200	-33500	-1194	-6250	-4031	-1528	-1087	-929.6	-858.1	-824.0	-804.6	-805.5	-808.3	-816.8	-828.0	-841.4
70	-437400	-59760	-19970	-9744	-5873	-1782	-1173	-968.1	-877.4	-833.0	-811.6	-803.3	-802.8	-807.7	-822.1	-827.0
80	-762000	-101600	-32780	-15320	-8654	-2198	-1304	-1019	-895.3	-833.2	-800.7	-784.3	-777.6	-777.1	-780.8	-787.4
90	-1257000	-165400	-52270	-23800	-13290	-2841	-1486	-1107	-933.3	-844.3	-794.8	-766.4	-750.3	-742.0	-738.9	-739.5
VALUES OF K_1 AT $\frac{3}{4}$ POINT																
10	-1627	-449.3	-300.7	-232.9	-201.7	-161.0	-156.7	-158.8	-163.1	-168.4	-174.3	-180.5	-187.0	-193.5	-200.2	207.0
20	-7014	-2093	-1172	-816.8	-684.1	-390.2	-344.7	-333.9	-334.3	-339.7	-347.6	-357.0	-367.4	-387.4	-389.9	-401.6
30	-22640	-4661	-2303	-1540	-1181	-653.3	-541.3	-505.1	-494.2	-494.3	-500.2	-509.8	-520.9	-533.7	-547.5	-562.2
40	-63290	-10440	-4297	-2553	-1824	-898.1	-715.0	-650.5	-624.4	-617.1	-618.2	-624.8	-634.7	-649.8	-660.1	-675.8
50	-148600	-22230	-8174	-4380	-2877	-1160	-870.6	-769.8	-726.5	-707.8	-702.0	-703.3	-709.4	-718.7	-730.2	-743.3
60	-306000	-43520	-15060	-7556	-4660	-1528	-1048	-888.5	-817.6	-782.7	-762.6	-758.9	-758.5	-762.3	-769.2	-778.1
70	-567200	-78820	-26350	-12710	-7519	-2065	-1286	-1038	-925.0	-865.5	-832.0	-813.0	-802.8	-798.4	-798.1	-800.6
80	-979400	-133900	-43840	-20620	-11810	-2857	-1633	-1245	-1071	-977.2	-921.3	-885.9	-862.8	-847.7	-838.0	-832.2
90	-1598000	-216100	-69780	-32290	-18280	-3983	-2110	-1529	-1272	-1133	-1048	-992.9	-954.3	-926.5	-906.0	-890.7
VALUES OF K_1 AT ABUTMENT																
10	-100.6	-111.9	-116.0	-118.4	-120.2	-127.9	-135.0	-142.1	-149.2	-156.3	-163.4	-170.4	-177.5	-184.6	-191.7	-198.8
20	-143.2	-165.0	-184.0	-198.6	-206.8	-233.0	-248.0	-262.4	-274.9	-288.7	-300.9	-314.6	-326.7	-340.5	-352.5	-366.3
30	-202.3	-210.3	-222.5	-236.1	-249.2	-296.6	-324.5	-345.5	-364.1	-381.6	-398.7	-415.5	-432.3	-449.0	-465.6	-482.3
40	-268.5	-270.8	-275.9	-282.8	-291.0	-333.7	-367.1	-392.9	-414.6	-434.4	-453.1	-471.3	-489.3	-507.0	-524.7	-542.4
50	-336.9	-337.6	-339.8	-343.1	-347.3	-374.5	-401.6	-424.7	-444.7	-462.7	-479.7	-496.0	-512.0	-527.9	-543.7	-559.5
60	-406.6	-407.2	-408.5	-410.5	-412.9	-429.4	-447.1	-462.9	-476.7	-489.2	-499.3	-512.2	-523.3	-534.5	-545.7	-557.0
70	-447.2	-478.3	-479.8	-481.7	-483.8	-495.9	-507.5	-517.0	-524.4	-530.4	-535.5	-540.3	-545.0	-549.8	-554.7	-559.9
80	-548.8	-550.8	-553.0	-555.3	-557.2	-570.0	-580.2	-587.0	-590.8	-592.4	-592.5	-591.7	-590.6	-589.4	-588.3	-587.3
90	-621.5	-624.5	-627.6	-630.8	-634.0	-649.2	-661.4	-669.5	-673.5	-674.3	-672.6	-669.3	-665.0	-660.1	-654.9	-648.6



Radial Deflection, in Ft

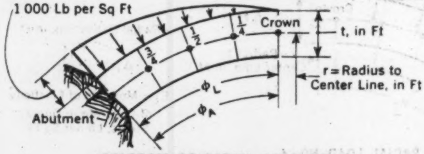
$$\Delta_r = \frac{K_1 r}{E_c}$$

E_c = Modulus of Elasticity of Concrete in Direct Stress, Lb per Sq Ft

DIAGRAM OF RADIAL LOAD NO. 4

TABLE 7(*a*).—FORCES AND MOMENTS FOR RADIAL LOAD No. 5

ϕ_A	VALUES OF t/r															
	.025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
VALUES OF h AT CROWN																
10	103.2	42.02	26.21	20.22	17.44	14.33	14.49	15.10	15.83	16.58	17.36	18.13	18.91	19.69	20.46	21.24
20	284.5	218.8	164.9	129.3	106.6	67.56	60.99	60.67	62.25	64.57	67.21	70.03	72.93	75.88	78.86	82.11
30	315.7	301.3	278.6	253.9	230.9	162.4	140.7	135.2	135.9	139.3	144.1	149.5	155.4	161.5	167.7	174.1
40	322.4	322.7	318.5	311.3	302.3	257.5	233.3	225.0	225.3	230.0	237.3	245.9	255.4	265.4	275.7	286.3
50	325.6	330.0	332.4	333.1	333.0	320.5	310.2	307.9	312.1	320.7	332.9	345.0	359.1	374.0	389.3	405.0
60	328.5	334.3	339.1	343.0	346.1	355.2	361.3	370.0	382.3	397.6	415.1	434.2	454.2	475.0	496.2	517.5
70	331.9	338.1	343.8	349.0	353.9	374.2	392.3	411.4	432.4	455.5	480.2	506.2	533.2	560.3	588.1	616.2
80	335.8	342.1	348.2	354.1	359.7	385.9	411.3	437.8	466.0	496.0	527.5	560.1	593.5	627.5	661.9	696.3
90	340.3	346.7	353.0	359.1	365.2	394.5	424.1	455.3	488.5	523.7	560.5	598.7	637.9	677.8	718.2	759.0
VALUES OF m AT CROWN																
10	0.921	1.462	1.748	1.967	2.157	2.912	3.511	4.028	4.492	4.923	5.328	5.716	6.091	6.455	6.810	7.160
20	-0.508	1.151	2.674	3.855	4.772	7.651	9.621	11.31	12.86	14.32	15.71	17.06	18.38	19.66	20.92	22.19
30	-2.648	-1.676	-0.230	1.403	3.029	9.370	13.51	16.73	19.54	22.12	24.56	26.92	29.22	31.46	33.68	35.84
40	-5.174	-4.779	-3.989	-2.894	-1.591	5.600	11.59	16.28	20.17	23.59	26.73	29.68	32.52	35.26	37.95	40.58
50	-8.267	-8.235	-7.939	-7.409	-6.682	-1.400	4.291	9.187	12.63	16.71	19.75	22.46	24.98	27.35	29.61	31.80
60	-12.00	-12.23	-12.28	-12.17	-11.93	-9.234	-5.596	-2.201	0.586	2.771	4.463	5.775	6.801	7.612	8.267	8.777
70	-16.42	-16.85	-17.17	-17.39	-17.51	-17.16	-16.12	-15.25	-14.93	-15.24	-16.11	-17.46	-19.26	-21.22	-23.49	-25.95
80	-21.54	-22.16	-22.70	-23.19	-23.61	-25.22	-26.64	-28.52	-31.18	-34.68	-38.94	-43.87	-49.35	-55.28	-62.02	-69.22
90	-27.39	-28.18	-28.94	-29.65	-30.33	-33.58	-37.15	-41.60	-47.20	-54.00	-61.93	-70.86	-80.62	-91.11	-102.2	-113.8
VALUES OF h AT ABUTMENT																
10	106.7	46.58	31.07	25.24	22.56	19.81	20.29	21.21	22.24	23.30	24.38	25.46	26.54	27.63	28.71	29.79
20	287.8	226.3	175.9	142.7	121.6	86.19	81.28	82.24	84.99	88.43	92.18	96.08	100.1	104.1	108.2	112.5
30	319.1	307.2	288.1	267.2	247.8	191.3	175.3	173.4	176.9	182.6	189.6	197.1	205.0	213.1	221.3	229.6
40	327.2	328.4	326.3	321.8	315.8	286.4	272.8	271.5	276.6	285.2	295.7	307.3	319.5	332.1	345.0	358.1
50	333.0	337.4	340.4	342.4	343.8	343.4	344.5	350.6	361.0	374.1	389.6	405.0	421.8	439.0	456.4	474.2
60	339.4	344.5	349.0	353.1	356.9	372.2	386.1	401.3	418.2	436.7	456.3	476.8	497.4	518.6	540.0	561.5
70	347.2	352.2	357.1	361.8	366.3	387.7	408.3	429.3	450.9	473.2	496.8	519.4	543.1	566.8	590.7	614.7
80	356.7	361.5	366.2	370.9	375.5	398.5	421.4	444.4	467.8	491.3	515.3	539.3	563.6	587.8	612.3	636.6
90	367.9	372.5	377.0	381.6	386.1	408.8	431.5	454.2	476.9	499.6	522.4	545.1	567.8	590.5	613.2	635.9
VALUES OF m AT ABUTMENT																
10	-2.644	-3.096	-3.113	-3.048	-2.964	-2.573	-2.288	-2.079	-1.920	-1.796	-1.695	-1.612	-1.543	-1.484	-1.433	-1.389
20	-3.788	-6.344	-8.324	-9.540	-10.24	-10.98	-10.67	-10.26	-9.878	-9.541	-9.247	-8.990	-8.765	-8.566	-8.389	-7.783
30	-5.983	-7.502	-9.659	-11.90	-13.93	-19.58	-21.16	-21.49	-21.41	-21.19	-20.92	-20.65	-20.39	-20.15	-19.92	-19.71
40	-10.02	-10.55	-11.71	-13.30	-15.10	-23.34	-27.97	-30.17	-31.17	-31.59	-31.72	-31.70	-31.61	-31.47	-31.32	-31.16
50	-15.67	-15.58	-15.95	-16.70	-17.56	-24.37	-30.00	-33.56	-36.24	-36.73	-36.97	-37.57	-37.72	-37.59	-37.48	-37.34
60	-22.90	-22.41	-22.24	-22.35	-22.69	-26.28	-30.38	-33.44	-35.31	-36.29	-36.66	-36.66	-36.42	-36.04	-35.59	-35.10
70	-31.79	-31.02	-30.48	-30.14	-29.96	-30.64	-32.12	-33.15	-33.39	-32.93	-31.33	-30.68	-29.15	-27.66	-26.08	-24.51
80	-42.44	-41.48	-40.68	-40.02	-39.53	-37.81	-36.74	-35.12	-32.93	-29.98	-26.72	-23.06	-19.42	-15.62	-12.41	-8.388
90	-55.01	-53.92	-52.95	-52.07	-51.26	-47.87	-44.54	-40.53	-35.64	-29.98	-23.76	-17.21	-10.51	-3.802	2.810	9.280
VALUES OF v AT ABUTMENT																
10	-70.22	-81.92	-85.76	-87.89	-89.46	-95.44	-100.8	-106.2	-111.5	-116.8	-122.1	-127.4	-132.7	-138.0	-143.3	-148.6
20	-77.63	-102.3	-122.9	-137.2	-147.1	-171.3	-184.3	-195.2	-205.5	-215.5	-225.4	-235.2	-245.0	-254.8	-264.6	-274.3
30	-101.2	-111.6	-126.1	-141.7	-156.4	-206.7	-233.5	-252.2	-267.9	-282.2	-295.8	-309.0	-322.1	-335.0	-347.9	-360.7
40	-132.1	-136.1	-142.9	-151.8	-161.8	-211.5	-248.0	-274.3	-295.0	-312.9	-329.2	-344.6	-359.5	-374.0	-388.3	-402.4
50	-165.0	-166.8	-170.0	-174.6	-179.9	-215.0	-248.5	-275.8	-298.2	-317.2	-333.4	-349.7	-364.5	-378.7	-392.5	-406.1
60	-198.9	-199.9	-201.7	-204.3	-207.6	-229.6	-254.1	-276.4	-295.6	-312.1	-326.8	-340.2	-352.7	-364.6	-376.0	-387.3
70	-233.4	-234.3	-235.7	-237.5	-239.7	-254.2	-270.9	-286.7	-300.5	-312.5	-321.1	-332.2	-340.5	-348.6	-356.2	-363.5
80	-268.6	-269.7	-271.1	-272.7	-274.6	-285.7	-297.7	-308.7	-317.9	-325.3	-331.3	-336.2	-340.3	-343.8	-346.9	-349.9
90	-304.3	-305.8	-307.5	-309.3	-311.2	-321.7	-331.9	-340.5	-347.1	-351.7	-354.6	-356.2	-356.8	-356.7	-356.1	-355.1



Thrust, Moment, Shear, in Lb, and Ft-Lb

$H = hr$

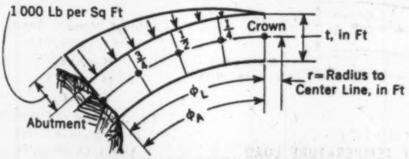
$M = mr^2$

$V = vr$

DIAGRAM OF RADIAL LOAD NO. 5

TABLE 7(*b*).—Radial Deflections for Radial Load No. 5

ϕ_A	VALUES OF t/r															
	.025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
	VALUES OF K_1 AT CROWN															
10	- 8417	- 1964	- 912.1	- 583.6	- 442.0	-267.0	-239.6	-234.2	-235.8	-240.5	-246.7	-253.9	-261.6	-269.8	-278.2	-286.9
20	- 15690	- 8320	- 4640	- 2936	- 2061	-842.2	-625.3	-557.6	-533.3	-526.8	-529.1	-536.2	-546.3	-558.4	-569.0	-585.8
30	- 10020	- 7305	- 6198	- 4794	- 3764	-1662	-1137	-948.5	-870.7	-833.7	-821.3	-820.0	-826.3	-838.0	-852.6	-869.6
40	83040	2256	- 3662	- 4163	- 3857	-2243	-1587	-1307	-1174	-1108	-1077	-1066	-1066	-1075	-1088	-1106
50	238200	23130	2931	- 1283	- 2187	-2297	-1788	-1514	-1385	-1290	-1246	-1232	-1230	-1236	-1249	-1267
60	525400	61380	15060	4197	587.7	-1865	-1700	-1520	-1407	-1343	-1309	-1295	-1294	-1302	-1317	-1336
70	1011000	125500	35240	13270	5551	-1001	-1421	-1342	-1296	-1265	-1250	-1248	-1255	-1269	-1289	-1312
80	1781000	226700	66860	27380	21480	335.5	-797.5	-1005	-1055	-1073	-1087	-1103	-1122	-1145	-1173	-1198
90	2949000	379400	114300	48430	24570	2260	7.967	-518.5	-699.4	-783.3	-834.5	-873.6	-910.0	-940.9	-988.3	-1006
	VALUES OF K_1 AT $\frac{1}{4}$ POINT															
10	- 7736	- 1827	- 862.1	- 558.9	- 427.5	-263.7	-238.0	-233.1	-235.0	-239.8	-246.1	-253.3	-261.1	-269.2	-277.7	-286.4
20	- 16810	- 7876	- 4346	- 2756	- 1945	-816.4	-614.4	-550.8	-528.4	-522.8	-525.6	-533.0	-543.5	-555.5	-566.8	-583.1
30	- 5330	- 8488	- 6219	- 4644	- 3604	-1598	-1106	-930.5	-859.3	-823.6	-812.1	-812.1	-819.2	-830.8	-856.0	-862.6
40	28840	- 4057	- 5242	- 4656	- 4002	-2183	-1547	-1281	-1155	-1093	-1064	-1053	-1054	-1063	-1076	-1094
50	101000	5855	- 2078	- 3296	- 2232	-2343	-1738	-1499	-1356	-1277	-1231	-1220	-1216	-1223	-1235	-1252
60	233500	23860	3744	- 6205	- 3500	-2153	-1851	-1549	-1421	-1349	-1310	-1293	-1290	-1296	-1309	-1326
70	456400	53760	13310	3759	5574	-1700	-1597	-1457	-1366	-1313	-1285	-1274	-1274	-1283	-1298	-1316
80	808100	100600	28150	10480	4269	-998.5	-1280	-1259	-1218	-1191	-1178	-1176	-1182	-1195	-1214	-1232
90	1338000	170700	50200	20390	9686	-10.23	-835.0	-972.3	-997.5	-1003	-1008	-1017	-1030	-1047	-1080	-1087
	VALUES OF K_1 AT $\frac{1}{2}$ POINT															
10	- 5676	- 1410	- 703.7	- 477.1	- 377.1	-249.9	-230.2	-227.6	-230.5	-235.9	-239.9	-250.1	-258.1	-266.4	-274.9	-283.7
20	- 16330	- 6193	- 3375	- 2176	- 1571	-723.7	-568.9	-521.2	-505.9	-504.0	-509.0	-517.8	-529.1	-542.1	-554.4	-570.3
30	- 28570	- 9291	- 5509	- 3875	- 2958	-1364	-988.9	-855.4	-803.6	-777.9	-772.1	-775.8	-785.2	-798.3	-814.0	-831.6
40	- 59350	- 13520	- 7138	- 4917	- 2638	-1891	-1323	-1161	-1053	-1018	-993.8	-992.9	-995.5	-1008	-1022	-1040
50	-125400	- 22070	- 9809	- 6157	- 4450	-2208	-1622	-1374	-1259	-1189	-1448	-1146	-1145	-1152	-1165	-1181
60	-248700	- 37980	-14670	- 8276	- 5673	-2439	-1768	-1494	-1358	-1285	-1247	-1229	-1224	-1228	-1238	-1253
70	-458900	- 65050	-22920	-11860	- 7556	-2717	-1878	-1562	-1408	-1325	-1280	-1257	-1247	-1248	-1254	-1266
80	-794400	-108300	-36100	-17570	-10560	-3138	-2014	-1622	-1437	-1338	-1282	-1251	-1236	-1230	-1233	-1239
90	-1306000	-174000	-56150	-26280	-15150	-3785	-2220	-1709	-1475	-1350	-1278	-1235	-1210	-1197	-1203	-1193
	VALUES OF K_1 AT $\frac{3}{4}$ POINT															
10	- 2632	- 775.1	- 447.1	- 336.3	- 285.7	-220.1	-212.0	-214.1	-219.4	-226.3	-234.0	-242.1	-250.6	-259.4	-268.2	-277.2
20	- 9142	- 3128	- 1785	- 1234	- 953.8	-550.3	-476.0	-456.6	-454.8	-460.7	-470.5	-482.6	-496.1	-510.6	-525.0	-541.2
30	- 23980	- 5792	- 3136	- 2186	-1704.8	-932.8	-757.1	-698.3	-682.1	-676.0	-682.2	-693.6	-708.2	-725.0	-742.9	-762.7
40	- 62370	- 11230	- 5043	- 3197	- 2386	-1250	-993.8	-898.6	-859.6	-846.1	-827.2	-853.9	-866.7	-882.9	-901.4	-921.6
50	-143100	- 22320	- 8665	- 4898	- 3336	-1521	-1172	-1043	-989.0	-962.2	-952.7	-957.4	-966.5	-979.8	-996.2	-1015
60	-291500	- 42420	-15130	- 7859	- 5018	-1852	-1337	-1158	-1078	-1039	-1022	-1017	-1020	-1028	-1040	-1055
70	-540500	- 75850	-25770	-12690	- 7683	-2338	-1541	-1285	-1166	-1106	-1074	-1058	-1051	-1052	-1057	-1065
80	-932700	-128100	-42310	-20150	-11770	-3058	-1847	-1457	-1283	-1190	-1137	-1105	-1086	-1076	-1071	-1071
90	-1522000	-206300	-66910	-31180	-17780	-4095	-2271	-1701	-1448	-1312	-1231	-1179	-1145	-1121	-1111	-1095
	VALUES OF K_1 AT ABUTMENT															
10	-125.3	-146.2	-153.1	-156.9	-159.7	-170.4	-180.0	-189.5	-199.0	-208.5	-218.0	-227.4	-238.9	-246.4	-255.9	-265.3
20	-138.6	-182.5	-219.3	-244.9	-262.6	-305.7	-329.0	-348.4	-366.8	-384.6	-402.3	-419.8	-437.3	-454.8	-472.3	-489.6
30	-180.7	-199.2	-225.2	-253.0	-278.5	-368.9	-415.8	-450.2	-480.5	-503.7	-526.6	-551.6	-573.5	-589.1	-619.5	-643.9
40	-235.8	-242.9	-255.1	-270.9	-288.7	-377.5	-442.7	-489.6	-526.6	-558.5	-587.7	-615.1	-641.7	-667.6	-693.1	-718.3
50	-294.6	-297.6	-303.5	-311.7	-321.1	-383.8	-443.5	-492.3	-532.2	-566.2	-595.1	-624.2	-650.6	-675.9	-700.6	-724.9
60	-355.1	-356.8	-360.1	-364.7	-370.5	-409.8	-453.5	-493.3	-527.6	-557.2	-583.4	-607.3	-629.5	-650.7	-671.2	-691.1
70	-416.7	-418.3	-420.8	-424.0	-427.9	-453.8	-483.6	-511.7	-536.4	-557.7	-576.4	-593.0	-607.8	-622.3	-635.8	-648.8
80	-479.4	-481.4	-483.9	-486.8	-490.1	-510.0	-531.4	-551.0	-567.3	-580.6	-591.3	-600.1	-607.3	-613.6	-619.1	-624.4
90	-543.1	-545.9	-548.9	-552.1	-555.6	-574.2	-592.4	-607.8	-619.5	-627.7	-633.0	-635.8	-636.9	-636.7	-635.6	-633.9



Radial Deflection, in Ft

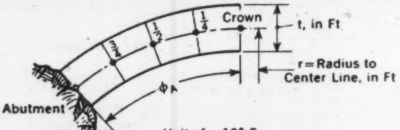
$$\Delta_r = \frac{K_1 r}{E_C}$$

E_C = Modulus of Elasticity of Concrete in Direct Stress, Lb per Sq Ft

DIAGRAM OF RADIAL LOAD NO. 5

TABLE 8(*a*).—FORCES AND MOMENTS FOR TEMPERATURE LOAD

ϕ_A	VALUES OF t/r															
	.025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
VALUES OF h AT CROWN																
10	$-.0^{6}710$	$-.0^{5}154$	$-.0^{5}211$	$-.0^{5}253$	$-.0^{5}286$	$-.0^{5}380$	$-.0^{5}426$	$-.0^{5}452$	$-.0^{5}470$	$-.0^{5}482$	$-.0^{5}491$	$-.0^{5}499$	$-.0^{5}504$	$-.0^{5}509$	$-.0^{5}513$	$-.0^{5}516$
20	$-.0^{6}147$	$-.0^{6}741$	$-.0^{5}153$	$-.0^{5}230$	$-.0^{5}298$	$-.0^{5}522$	$-.0^{5}645$	$-.0^{5}722$	$-.0^{5}776$	$-.0^{5}816$	$-.0^{5}846$	$-.0^{5}870$	$-.0^{5}890$	$-.0^{5}906$	$-.0^{5}920$	$-.0^{5}932$
30	$-.0^{7}340$	$-.0^{6}227$	$-.0^{6}618$	$-.0^{5}116$	$-.0^{5}178$	$-.0^{5}476$	$-.0^{5}683$	$-.0^{5}823$	$-.0^{5}924$	$-.0^{4}100$	$-.0^{4}106$	$-.0^{4}111$	$-.0^{4}115$	$-.0^{4}118$	$-.0^{4}121$	$-.0^{4}123$
40	$-.0^{7}112$	$-.0^{7}810$	$-.0^{6}243$	$-.0^{6}507$	$-.0^{6}863$	$-.0^{5}335$	$-.0^{5}579$	$-.0^{5}772$	$-.0^{5}921$	$-.0^{4}104$	$-.0^{4}113$	$-.0^{4}120$	$-.0^{4}127$	$-.0^{4}132$	$-.0^{4}136$	$-.0^{4}140$
50	$-.0^{8}465$	$-.0^{7}347$	$-.0^{6}108$	$-.0^{6}236$	$-.0^{6}423$	$-.0^{5}208$	$-.0^{5}425$	$-.0^{5}631$	$-.0^{5}808$	$-.0^{5}954$	$-.0^{4}108$	$-.0^{4}118$	$-.0^{4}126$	$-.0^{4}134$	$-.0^{4}140$	$-.0^{4}145$
60	$-.0^{8}224$	$-.0^{7}170$	$-.0^{7}543$	$-.0^{6}121$	$-.0^{6}222$	$-.0^{5}125$	$-.0^{5}289$	$-.0^{5}473$	$-.0^{5}649$	$-.0^{5}807$	$-.0^{5}946$	$-.0^{4}107$	$-.0^{4}117$	$-.0^{4}127$	$-.0^{4}134$	$-.0^{4}141$
70	$-.0^{8}120$	$-.0^{8}922$	$-.0^{7}298$	$-.0^{7}673$	$-.0^{6}125$	$-.0^{6}766$	$-.0^{5}193$	$-.0^{5}339$	$-.0^{5}495$	$-.0^{5}645$	$-.0^{5}785$	$-.0^{5}912$	$-.0^{4}103$	$-.0^{4}113$	$-.0^{4}122$	$-.0^{4}130$
80	$-.0^{9}696$	$-.0^{8}537$	$-.0^{7}175$	$-.0^{7}398$	$-.0^{7}748$	$-.0^{6}483$	$-.0^{5}129$	$-.0^{5}240$	$-.0^{5}366$	$-.0^{5}498$	$-.0^{5}627$	$-.0^{5}750$	$-.0^{5}863$	$-.0^{5}968$	$-.0^{4}106$	$-.0^{4}115$
90	$-.0^{9}425$	$-.0^{8}330$	$-.0^{7}108$	$-.0^{7}248$	$-.0^{7}468$	$-.0^{6}314$	$-.0^{6}873$	$-.0^{5}169$	$-.0^{5}268$	$-.0^{5}377$	$-.0^{5}489$	$-.0^{5}600$	$-.0^{5}706$	$-.0^{5}807$	$-.0^{5}901$	$-.0^{5}988$
VALUES OF m AT CROWN																
10	$.0^{8}401$	$.0^{8}946$	$.0^{7}140$	$.0^{7}178$	$.0^{7}212$	$.0^{7}338$	$.0^{7}421$	$.0^{7}480$	$.0^{7}525$	$.0^{7}559$	$.0^{7}587$	$.0^{7}610$	$.0^{7}629$	$.0^{7}646$	$.0^{7}659$	$.0^{7}672$
20	$.0^{8}314$	$.0^{7}167$	$.0^{7}361$	$.0^{7}564$	$.0^{7}758$	$.0^{6}154$	$.0^{6}211$	$.0^{6}225$	$.0^{6}291$	$.0^{6}321$	$.0^{6}346$	$.0^{6}367$	$.0^{6}386$	$.0^{6}402$	$.0^{6}416$	$.0^{6}429$
30	$.0^{8}159$	$.0^{7}110$	$.0^{7}310$	$.0^{7}600$	$.0^{7}949$	$.0^{6}286$	$.0^{6}449$	$.0^{6}582$	$.0^{6}692$	$.0^{6}786$	$.0^{6}867$	$.0^{6}938$	$.0^{5}100$	$.0^{5}106$	$.0^{5}111$	$.0^{5}115$
40	$.0^{9}914$	$.0^{8}679$	$.0^{7}209$	$.0^{7}447$	$.0^{7}779$	$.0^{6}334$	$.0^{6}625$	$.0^{6}890$	$.0^{5}112$	$.0^{5}132$	$.0^{5}150$	$.0^{5}166$	$.0^{5}180$	$.0^{5}192$	$.0^{5}204$	$.0^{5}214$
50	$.0^{9}581$	$.0^{8}443$	$.0^{7}141$	$.0^{7}315$	$.0^{7}573$	$.0^{6}306$	$.0^{6}673$	$.0^{5}106$	$.0^{5}143$	$.0^{5}176$	$.0^{5}207$	$.0^{5}234$	$.0^{5}259$	$.0^{5}282$	$.0^{5}302$	$.0^{5}322$
60	$.0^{9}396$	$.0^{8}306$	$.0^{8}991$	$.0^{7}225$	$.0^{7}419$	$.0^{6}254$	$.0^{6}626$	$.0^{5}108$	$.0^{5}155$	$.0^{5}201$	$.0^{5}244$	$.0^{5}285$	$.0^{5}322$	$.0^{5}356$	$.0^{5}388$	$.0^{5}418$
70	$.0^{9}282$	$.0^{8}220$	$.0^{8}719$	$.0^{7}165$	$.0^{7}311$	$.0^{6}203$	$.0^{6}540$	$.0^{6}997$	$.0^{5}152$	$.0^{5}206$	$.0^{5}259$	$.0^{5}311$	$.0^{5}359$	$.0^{5}406$	$.0^{5}449$	$.0^{5}489$
80	$.0^{9}208$	$.0^{8}163$	$.0^{8}535$	$.0^{7}124$	$.0^{7}235$	$.0^{6}160$	$.0^{6}450$	$.0^{6}874$	$.0^{5}139$	$.0^{5}196$	$.0^{5}255$	$.0^{5}314$	$.0^{5}371$	$.0^{5}426$	$.0^{5}480$	$.0^{5}530$
90	$.0^{9}156$	$.0^{8}123$	$.0^{8}406$	$.0^{8}941$	$.0^{7}180$	$.0^{6}127$	$.0^{6}368$	$.0^{6}741$	$.0^{5}122$	$.0^{5}178$	$.0^{5}237$	$.0^{5}299$	$.0^{5}361$	$.0^{5}423$	$.0^{5}482$	$.0^{5}540$
VALUES OF h AT ABUTMENT																
10	$-.0^{6}700$	$-.0^{5}151$	$-.0^{5}208$	$-.0^{5}249$	$-.0^{5}282$	$-.0^{5}375$	$-.0^{5}419$	$-.0^{5}445$	$-.0^{5}463$	$-.0^{5}475$	$-.0^{5}484$	$-.0^{5}491$	$-.0^{5}497$	$-.0^{5}501$	$-.0^{5}505$	$-.0^{5}508$
20	$-.0^{6}138$	$-.0^{6}697$	$-.0^{5}144$	$-.0^{5}216$	$-.0^{5}280$	$-.0^{5}490$	$-.0^{5}606$	$-.0^{5}679$	$-.0^{5}729$	$-.0^{5}767$	$-.0^{5}795$	$-.0^{5}818$	$-.0^{5}836$	$-.0^{5}852$	$-.0^{5}864$	$-.0^{5}875$
30	$-.0^{7}295$	$-.0^{6}197$	$-.0^{6}535$	$-.0^{5}100$	$-.0^{5}154$	$-.0^{5}412$	$-.0^{5}591$	$-.0^{5}713$	$-.0^{5}800$	$-.0^{5}866$	$-.0^{5}917$	$-.0^{5}958$	$-.0^{5}992$	$-.0^{4}102$	$-.0^{4}104$	$-.0^{4}106$
40	$-.0^{8}858$	$-.0^{7}620$	$-.0^{6}186$	$-.0^{6}388$	$-.0^{6}661$	$-.0^{5}257$	$-.0^{5}444$	$-.0^{5}592$	$-.0^{5}705$	$-.0^{5}794$	$-.0^{5}864$	$-.0^{5}922$	$-.0^{5}969$	$-.0^{4}101$	$-.0^{4}104$	$-.0^{4}107$
50	$-.0^{8}299$	$-.0^{7}223$	$-.0^{7}697$	$-.0^{6}152$	$-.0^{6}272$	$-.0^{5}133$	$-.0^{5}273$	$-.0^{5}406$	$-.0^{5}519$	$-.0^{5}613$	$-.0^{5}691$	$-.0^{5}757$	$-.0^{5}812$	$-.0^{5}859$	$-.0^{5}899$	$-.0^{5}935$
60	$-.0^{8}112$	$-.0^{8}851$	$-.0^{7}271$	$-.0^{7}606$	$-.0^{6}111$	$-.0^{6}625$	$-.0^{5}145$	$-.0^{5}237$	$-.0^{5}325$	$-.0^{5}404$	$-.0^{5}473$	$-.0^{5}533$	$-.0^{5}585$	$-.0^{5}630$	$-.0^{5}670$	$-.0^{5}705$
70	$-.0^{9}412$	$-.0^{8}316$	$-.0^{7}102$	$-.0^{7}230$	$-.0^{7}428$	$-.0^{6}262$	$-.0^{6}660$	$-.0^{5}116$	$-.0^{5}169$	$-.0^{5}221$	$-.0^{5}270$	$-.0^{5}312$	$-.0^{5}351$	$-.0^{5}385$	$-.0^{5}416$	$-.0^{5}444$
80	$-.0^{9}121$	$-.0^{9}933$	$-.0^{8}303$	$-.0^{8}692$	$-.0^{7}130$	$-.0^{7}838$	$-.0^{6}224$	$-.0^{6}416$	$-.0^{6}636$	$-.0^{6}865$	$-.0^{5}109$	$-.0^{5}130$	$-.0^{5}150$	$-.0^{5}168$	$-.0^{5}185$	$-.0^{5}200$
90	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
VALUES OF m AT ABUTMENT																
10	$-.0^{8}678$	$-.0^{7}139$	$-.0^{7}181$	$-.0^{7}206$	$-.0^{7}222$	$-.0^{7}240$	$-.0^{7}226$	$-.0^{7}207$	$-.0^{7}189$	$-.0^{7}173$	$-.0^{7}159$	$-.0^{7}147$	$-.0^{7}137$	$-.0^{7}128$	$-.0^{7}120$	$-.0^{7}113$
20	$-.0^{8}573$	$-.0^{7}280$	$-.0^{7}564$	$-.0^{7}823$	$-.0^{6}104$	$-.0^{6}161$	$-.0^{6}178$	$-.0^{6}180$	$-.0^{6}177$	$-.0^{6}172$	$-.0^{6}165$	$-.0^{6}158$	$-.0^{6}151$	$-.0^{6}145$	$-.0^{6}138$	$-.0^{6}133$
30	$-.0^{8}296$	$-.0^{7}194$	$-.0^{7}518$	$-.0^{7}954$	$-.0^{6}144$	$-.0^{6}352$	$-.0^{6}466$	$-.0^{6}521$	$-.0^{6}546$	$-.0^{6}553$	$-.0^{6}552$	$-.0^{6}544$	$-.0^{6}533$	$-.0^{6}521$	$-.0^{6}508$	$-.0^{6}494$
40	$-.0^{8}171$	$-.0^{7}122$	$-.0^{7}360$	$-.0^{7}740$	$-.0^{6}124$	$-.0^{6}450$	$-.0^{6}730$	$-.0^{6}917$	$-.0^{5}103$	$-.0^{5}110$	$-.0^{5}114$	$-.0^{5}116$	$-.0^{5}116$	$-.0^{5}116$	$-.0^{5}115$	$-.0^{5}114$
50	$-.0^{8}108$	$-.0^{8}796$	$-.0^{7}246$	$-.0^{7}530$	$-.0^{7}937$	$-.0^{6}435$	$-.0^{6}845$	$-.0^{5}119$	$-.0^{5}146$	$-.0^{5}165$	$-.0^{5}178$	$-.0^{5}186$	$-.0^{5}192$	$-.0^{5}196$	$-.0^{5}197$	$-.0^{5}198$
60	$-.0^{9}727$	$-.0^{8}546$	$-.0^{7}172$	$-.0^{7}381$	$-.0^{7}691$	$-.0^{6}371$	$-.0^{6}822$	$-.0^{5}129$	$-.0^{5}169$	$-.0^{5}203$	$-.0^{5}229$	$-.0^{5}248$	$-.0^{5}263$	$-.0^{5}274$	$-.0^{5}282$	$-.0^{5}287$
70	$-.0^{9}510$	$-.0^{8}387$	$-.0^{7}124$	$-.0^{7}278$	$-.0^{7}512$	$-.0^{6}301$	$-.0^{6}730$	$-.0^{5}124$	$-.0^{5}174$	$-.0^{5}219$	$-.0^{5}258$	$-.0^{5}289$	$-.0^{5}315$	$-.0^{5}336$	$-.0^{5}352$	$-.0^{5}365$
80	$-.0^{9}367$	$-.0^{8}281$	$-.0^{8}908$	$-.0^{7}206$	$-.0^{7}383$	$-.0^{6}239$	$-.0^{6}616$	$-.0^{5}111$	$-.0^{5}164$	$-.0^{5}216$	$-.0^{5}264$	$-.0^{5}306$	$-.0^{5}342$	$-.0^{5}373$	$-.0^{5}399$	$-.0^{5}421$
90	$-.0^{9}269$	$-.0^{8}207$	$-.0^{8}673$	$-.0^{7}154$	$-.0^{7}288$	$-.0^{6}187$	$-.0^{6}505$	$-.0^{6}948$	$-.0^{5}146$	$-.0^{5}200$	$-.0^{5}252$	$-.0^{5}301$	$-.0^{5}345$	$-.0^{5}384$	$-.0^{5}419$	$-.0^{5}448$
VALUES OF v AT ABUTMENT																
10	$-.0^{6}123$	$-.0^{6}267$	$-.0^{6}366$	$-.0^{6}440$	$-.0^{6}497$	$-.0^{6}660$	$-.0^{6}739$	$-.0^{6}785$	$-.0^{6}816$	$-.0^{6}837$	$-.0^{6}853$	$-.0^{6}866$	$-.0^{6}876$	$-.0^{6}884$	$-.0^{6}891$	$-.0^{6}897$
20	$-.0^{7}503$	$-.0^{6}254$	$-.0^{6}524$	$-.0^{6}786$	$-.0^{5}102$	$-.0^{5}178$	$-.0^{5}207$	$-.0^{5}247$	$-.0^{5}265$	$-.0^{5}279$	$-.0^{5}289$	$-.0^{5}298$	$-.0^{5}304$	$-.0^{5}310$	$-.0^{5}315$	$-.0^{5}319$
30	$-.0^{7}170$	$-.0^{6}114$	$-.0^{6}309$	$-.0^{6}580$	$-.0^{6}891$	$-.0^{5}238$	$-.0^{5}341$	$-.0^{5}412$	$-.0^{5}462$	$-.0^{5}500$	$-.0^{5}529$	$-.0^{5}553$	$-.0^{5}573$	$-.0^{5}589$	$-.0^{5}603$	$-.0^{5}615$
40	$-.0^{8}720$	$-.0^{7}521$	$-.0^{6}156$	$-.0^{6}326$	$-.0^{6}555$	$-.0^{5}215$	$-.0^{5}372$	$-.0^{5}496$	$-.0^{5}592$	$-.0^{5}666$	$-.0^{5}725$	$-.0^{5}773$	$-.0^{5}813$	$-.0^{5}847$	$-.0^{5}876$	$-.0^{5}901$
50	$-.0^{8}356$	$-.0^{7}266$	$-.0^{7}830$	$-.0^{6}181$	$-.0^{6}324$	$-.0^{5}159$	$-.0^{5}326$	$-.0^{5}484$	$-.0^{5}619$	$-.0^{5}731$	$-.0^{5}824$	$-.0^{5}902$	$-.0^{5}968$	$-.0^{4}102$	$-.0^{4}107$	$-.0^{4}111$
60	$-.0^{8}194$	$-.0^{7}147$	$-.0^{7}470$	$-.0^{6}105$	$-.0^{6}192$	$-.0^{5}108$	$-.0^{5}251$	$-.0^{5}410$	$-.0^{5}562$	$-.0^{5}699$	$-.0^{5}819$	$-.0^{5}923$	$-.0^{4}101$	$-.0^{4}110$	$-.0^{4}116$	$-.0^{4}122$
70	$-.0^{8}113$	$-.0^{8}867$	$-.0^{7}280$	$-.0^{7}632$	$-.0^{6}118$	$-.0^{6}720$	$-.0^{5}181$	$-.0^{5}319$	$-.0^{5}465$	$-.0^{5}606$	$-.0^{5}741$	$-.0^{5}857$	$-.0^{5}964$	$-.0^{4}106$	$-.0^{4}114$	$-.0^{4}122$
80	$-.0^{9}685$	$-.0^{8}529$	$-.0^{7}172$	$-.0^{7}392$	$-.0^{7}737$	$-.0^{6}476$	$-.0^{5}127$	$-.0^{5}236$	$-.0^{5}361$	$-.0^{5}491$	$-.0^{5}618$	$-.0^{5}738$	$-.0^{5}850$	$-.0^{5}953$	$-.0^{4}105$	$-.0^{4}113$
90	$-.0^{9}425$	$-.0^{8}330$	$-.0^{7}108$	$-.0^{7}248$	$-.0^{7}468$	$-.0^{6}314$	$-.0^{6}873$	$-.0^{5}169$	$-.0^{5}268$	$-.0^{5}377$	$-.0^{5}489$	$-.0^{5}600$	$-.0^{5}706$	$-.0^{5}807$	$-.0^{5}901$	$-.0^{5}988$



Unit of $-10°$ F

DISTRIBUTION OF TEMPERATURE LOAD

UNIFORM THROUGHOUT ARCH

Thrust, Moment, Shear
in Lb, and Ft-Lb

$H = h r E_c$

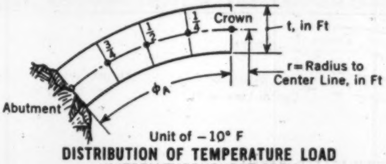
$M = m r^2 E_c$

$V = v r E_c$

E_c = Modulus of Elasticity of Concrete in Direct Stress, Lb per Sq Ft

TABLE 8(*b*).—Radial Deflections for Temperature Load

ϕ_A	VALUES OF t/r															
	.025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
VALUES OF K_1 AT CROWN																
10	-5.47	-1.97	-1.08	-.69	-0.43	-0.33	-0.28	-0.25	-0.24	-0.23	-.225	-0.22	-0.22	-0.22	-.215	-0.21
20	-15.90	-11.45	-8.07	-5.91	-4.52	-2.09	-1.56	-1.29	-1.13	-1.06	-1.01	-0.97	-0.95	-0.93	-9.15	0.90
30	-17.93	-16.30	-14.37	-12.48	-10.49	-5.93	-4.22	-3.31	-2.85	-2.62	-2.45	-2.33	-2.23	-2.17	-2.12	-2.07
40	-18.41	-17.71	-16.82	-15.82	-14.80	-10.31	-7.71	-6.24	-5.39	-4.85	-4.48	-4.22	-4.03	-3.88	-3.78	-3.67
50	-18.61	-18.23	-17.76	-17.22	-16.67	-13.54	-11.05	-9.37	-8.22	-7.43	-6.87	-6.46	-6.15	-5.91	-5.72	-5.56
60	-18.75	-18.50	-18.21	-17.88	-17.53	-15.52	-13.60	-12.04	-10.86	-9.96	-9.29	-8.77	-8.37	-8.05	-7.79	-7.58
70	-18.86	-18.68	-18.48	-18.26	-18.02	-16.69	-15.32	-14.05	-13.00	-12.15	-11.47	-10.91	-10.47	-10.11	-9.82	-9.56
80	-18.97	-18.83	-18.68	-18.53	-18.36	-17.43	-16.43	-15.48	-14.63	-13.90	-13.28	-12.76	-12.32	-11.96	-11.66	-11.39
90	-19.08	-18.97	-18.86	-18.74	-18.62	-17.94	-17.22	-16.50	-15.84	-15.25	-14.74	-14.28	-13.88	-13.55	-13.27	-13.01
VALUES OF K_1 AT $\frac{1}{4}$ POINT																
10	-4.88	-1.78	-0.99	-0.69	-0.52	-0.32	-0.28	-0.25	-0.24	-0.24	-0.23	-0.23	-0.22	-0.22	-0.22	-0.22
20	-14.08	-10.21	-07.25	-5.35	-4.17	-1.97	-1.46	-1.25	-1.14	-1.05	-1.00	-0.98	-0.96	-0.94	-.925	-0.91
30	-15.82	-14.46	-12.81	-11.18	-9.61	-5.47	-3.86	-3.16	-2.78	-2.55	-2.41	-2.30	-2.22	-2.16	-2.12	-2.08
40	-16.21	-15.65	-14.92	-14.08	-13.20	-9.38	-7.15	-5.86	-5.11	-4.65	-4.33	-4.10	-3.94	-3.81	-3.72	-3.63
50	-16.36	-16.07	-15.70	-15.26	-14.79	-12.22	-10.10	-8.67	-7.70	-7.02	-6.54	-6.19	-5.94	-5.73	-5.57	-5.43
60	-16.44	-16.26	-16.05	-15.80	-15.52	-13.90	-12.32	-11.03	-10.04	-9.29	-8.73	-8.30	-7.97	-7.70	-7.48	-7.31
70	-16.51	-16.38	-16.24	-16.08	-15.90	-14.87	-13.77	-12.75	-11.89	-11.20	-10.84	-10.20	-9.84	-9.55	-9.31	-9.10
80	-16.57	-16.47	-16.37	-16.26	-16.14	-15.45	-14.69	-13.94	-13.27	-12.69	-12.20	-11.79	-11.45	-11.16	-10.91	-10.72
90	-16.62	-16.55	-16.48	-16.40	-16.31	-15.82	-15.29	-14.75	-14.25	-13.80	-13.40	-13.06	-12.76	-12.51	-12.29	-12.10
VALUES OF K_1 AT $\frac{1}{2}$ POINT																
10	-3.32	-1.29	-.756	-.549	-0.45	-.302	-0.28	-0.26	-0.25	-0.25	-0.24	-0.24	-0.24	-0.24	-0.24	-0.24
20	-9.28	-6.93	-5.07	-3.85	-3.10	-1.63	-1.29	-1.14	-1.08	-1.04	-1.01	-0.994	-0.98	-.972	-0.96	-.958
30	-10.30	-9.60	-8.68	-7.72	-6.85	-4.20	-3.23	-2.74	-2.49	-2.36	-2.27	-2.22	-2.17	-2.14	-2.12	-2.10
40	-10.47	-10.26	-9.92	-9.50	-9.02	-6.88	-5.58	-4.79	-4.33	-4.08	-3.90	-3.77	-3.68	-3.62	-3.56	-3.52
50	-10.49	-10.43	-10.31	-10.14	-9.90	-8.67	-7.58	-6.77	-6.23	-5.88	-5.62	-5.44	-5.31	-5.21	-5.13	-5.07
60	-10.48	-10.47	-10.43	-10.36	-10.26	-9.62	-8.92	-8.28	-7.80	-7.45	-7.18	-6.99	-6.83	-6.72	-6.62	-6.55
70	-10.46	-10.46	-10.45	-10.43	-10.39	-10.08	-9.68	-9.27	-8.92	-8.64	-8.43	-8.25	-8.11	-8.00	-7.92	-7.85
80	-10.42	-10.43	-10.44	-10.43	-10.42	-10.29	-10.09	-9.86	-9.65	-9.46	-9.30	-9.18	-9.08	-9.00	-8.93	-8.88
90	-10.38	-10.40	-10.40	-10.41	-10.41	-10.37	-10.28	-10.17	-10.06	-9.96	-9.87	-9.81	-9.76	-9.71	-9.68	-9.65
VALUES OF K_1 AT $\frac{3}{4}$ POINT																
10	-1.38	-.633	-.432	-.353	-0.30	-0.27	-0.27	-0.27	-0.27	-0.27	-0.27	-0.27	-0.27	-0.27	-0.27	-0.27
20	-3.50	-2.87	-2.29	-1.89	-1.62	-1.15	-1.07	-1.02	-1.01	-1.01	-1.01	-1.02	-1.02	-1.03	-1.03	-1.03
30	-3.74	-3.72	-3.57	-3.37	-3.17	-2.47	-2.20	-2.13	-2.10	-2.09	-2.09	-2.10	-2.10	-2.11	-2.12	-2.13
40	-3.72	-3.83	-3.88	-3.88	-3.84	-3.58	-3.39	-3.30	-3.26	-3.26	-3.27	-3.29	-3.31	-3.33	-3.35	-3.37
50	-3.67	-3.79	-3.89	-3.96	-4.01	-4.13	-4.17	-4.18	-4.21	-4.27	-4.32	-4.37	-4.41	-4.46	-4.50	-4.54
60	-3.61	-3.72	-3.82	-3.91	-3.99	-4.29	-4.50	-4.66	-4.80	-4.93	-5.05	-5.15	-5.24	-5.33	-5.40	-5.48
70	-3.55	-3.65	-3.74	-3.83	-3.91	-4.26	-4.55	-4.81	-5.04	-5.25	-5.44	-5.60	-5.74	-5.88	-6.00	-6.11
80	-3.49	-3.57	-3.65	-3.73	-3.81	-4.15	-4.46	-4.76	-5.03	-5.29	-5.52	-5.73	-5.92	-6.10	-6.26	-6.41
90	-3.43	-3.50	-3.57	-3.63	-3.69	-4.01	-4.31	-4.60	-4.87	-5.15	-5.41	-5.65	-5.87	-6.07	-6.25	-6.43
VALUES OF K_1 AT ABUTMENT																
10	-.0 440	-.0 952	-.131	-.157	-.177	-.236	-.264	-.280	-.291	-.299	-0.304	-0.309	-.313	-.316	-.318	-.32
20	-.0 180	-.0 905	-.187	-.281	-.364	-.637	-.739	-.882	-.946	-.996	-1.03	-1.06	-1.09	-1.11	-1.12	-1.14
30	$-.0^{2}607$	-.0 406	-.110	-.207	-.318	-.850	-1.22	-1.47	-1.65	-1.78	-1.89	-1.97	-2.05	-2.10	-2.15	-2.19
40	$-.0^{2}257$	-.0 186	-.0 558	-.116	-.198	-.769	-1.33	-1.77	-2.11	-2.38	-2.59	-2.76	-2.90	-3.05	-3.13	-3.22
50	$-.0^{2}127$	$-.0^{2}949$	-.0 296	-.0 647	-.116	-.368	-1.16	-1.73	-2.21	-2.61	-2.94	-3.22	-3.46	-3.65	-3.82	-3.98
60	$-.0^{3}694$	$-.0^{2}526$	-.0 168	-.0 375	-.0 685	-.387	-.896	-1.46	-2.01	-2.50	-2.92	-3.30	-3.61	-3.90	-4.14	-4.36
70	$-.0^{3}404$	$-.0^{2}309$	$-.0^{2}999$	-.0 226	-.0 421	-.257	-.646	-1.14	-1.66	-2.16	-2.65	-3.06	-3.44	-3.78	-4.07	-4.36
80	$-.0^{3}245$	$-.0^{2}189$	$-.0^{2}614$	-.0 140	-.0 263	-.170	-.453	-.842	-1.29	-1.75	-2.21	-2.64	-3.03	-3.40	-3.75	-4.04
90	$-.0^{3}152$	$-.0^{2}118$	$-.0^{2}385$	$-.0^{2}884$	.0 167	-.112	-.312	-.603	-.957	-1.35	-1.75	-2.14	-2.52	-2.88	-3.22	-3.53



Unit of −10° F

DISTRIBUTION OF TEMPERATURE LOAD
UNIFORM THROUGHOUT ARCH

Radial Deflection, in Ft
$\Delta_r = K_1 r c T$
C=Coefficient of Thermal Expansion of Concrete per Degree Fahrenheit Change in Temperature
T=Temperature Change in Degrees Fahrenheit ÷ (−10)
Example (Δ_r at Crown):
$\phi_A = 10°$; $\frac{t}{r} = 0.125$; T=25° F

$\Delta_r = (-0.43)\ (r)\ (c)\ (-2.5)$

TABLE 9(*a*).—FORCES AND MOMENTS FOR CONCENTRATED RADIAL LOAD AT ABUTMENT

ϕ_A	VALUES OF t/r															
	.025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
	VALUES OF h AT CROWN															
10	25.36	54.83	75.29	90.41	102.1	135.8	151.9	161.4	167.7	172.1	175.4	178.0	180.1	181.7	183.1	184.3
20	5.251	26.47	54.72	82.09	106.2	186.3	230.1	257.9	276.5	291.3	302.1	310.8	317.7	323.5	328.4	332.6
30	1.214	8.110	22.07	41.40	63.64	170.0	243.8	293.9	329.9	356.9	378.0	395.0	408.9	420.5	430.4	438.8
40	.4000	2.891	8.685	18.10	30.83	119.6	206.9	275.7	328.6	369.9	402.8	429.5	451.7	470.4	486.4	500.2
50	.1660	1.238	3.870	8.443	15.10	74.13	151.7	225.4	236.6	340.7	384.0	420.3	450.9	477.0	499.6	519.2
60	.08014	.6079	1.938	4.325	7.929	44.65	103.4	168.9	231.8	288.2	337.6	380.5	417.7	450.1	478.4	503.4
70	.04297	.3293	1.063	2.403	4.469	27.34	68.89	121.2	176.5	230.3	281.3	325.5	366.1	402.3	434.7	463.7
80	.02484	.1918	.6237	1.423	2.671	17.24	46.03	85.53	130.8	177.8	223.9	267.6	308.2	345.5	379.6	410.6
90	.01517	.1178	.3852	.8842	1.671	11.19	31.15	60.30	95.81	134.7	174.7	214.2	252.2	288.0	321.6	352.8
	VALUES OF m AT CROWN															
10	-.1432	-.3379	-.4988	-.6368	-.7581	-1.207	-1.502	-1.713	-1.873	-1.997	-2.096	-2.178	-2.246	-2.304	-2.354	-2.398
20	-.1122	-.5949	-1.287	-2.013	-2.705	-5.498	-7.529	-9.108	-10.38	-11.44	-12.34	-13.10	-13.77	-14.35	-14.86	-15.32
30	-.05687	-.3935	-1.123	-2.141	-3.387	-10.20	-16.02	-20.76	-24.71	-28.06	-30.95	-33.49	-35.74	-37.73	-39.53	-41.15
40	-.03263	-.2424	-.7469	-1.595	-2.779	-11.92	-22.32	-31.77	-40.01	-47.20	-53.51	-59.11	-64.13	-68.65	-72.75	-76.49
50	-.02074	-.1582	-.5048	-1.123	-2.047	-10.94	-24.02	-37.87	-41.83	-62.98	-73.79	-83.60	-92.49	-100.6	-108.0	-114.8
60	-.01413	-.1091	-.3540	-.8032	-1.496	-9.064	-22.34	-38.50	-55.37	-71.77	-87.24	-101.6	-114.9	-127.2	-138.6	-149.2
70	-.01008	-.07842	-.2568	-.5891	-1.111	-7.241	-19.28	-35.61	-54.17	-73.45	-92.89	-110.9	-128.3	-144.8	-160.2	-174.7
80	$-.0^{2}7420$	-.05803	-.1911	-.4415	-.8388	-5.725	-16.06	-31.19	-49.62	-69.91	-90.93	-111.9	-132.5	-152.2	-171.2	-189.2
90	$-.0^{2}5577$	-.04378	-.1448	-.3360	-.6418	-4.517	-13.14	-26.47	-43.60	-63.37	-84.71	-106.8	-129.0	-150.8	-172.2	-192.8
	VALUES OF h AT ABUTMENT															
10	24.98	53.99	74.15	89.04	100.6	133.7	149.6	159.0	165.2	169.5	172.8	175.3	177.3	179.0	180.4	181.5
20	4.935	24.85	51.42	77.14	99.84	175.1	216.2	242.3	259.8	273.7	283.9	292.0	298.6	304.0	308.6	312.5
30	1.052	7.024	19.11	35.85	55.11	147.2	211.1	254.5	285.7	309.1	327.4	342.0	354.1	364.2	372.7	380.0
40	.3064	2.215	6.653	13.87	23.61	91.62	158.5	211.2	251.8	283.4	308.5	329.0	346.0	360.4	372.6	383.2
50	.1067	.7960	2.488	5.427	9.703	47.65	97.52	144.9	152.1	219.0	246.8	270.2	289.8	306.6	321.1	333.7
60	.04007	.3040	.9691	2.163	3.964	22.33	51.70	84.46	115.9	144.1	168.8	190.3	208.9	225.0	239.2	251.7
70	.01470	.1126	.3634	.8218	1.528	9.350	23.56	41.44	60.38	78.78	96.23	111.3	125.2	137.6	148.7	158.6
80	$.0^{2}4314$	.03330	.1083	.2470	.4637	2.993	7.993	14.85	22.72	30.88	38.89	46.47	53.51	59.99	65.91	71.31
90	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	VALUES OF m AT ABUTMENT															
10	.2421	.4950	.6450	.7367	.7935	.8559	.8057	.7388	.6752	.6185	.5692	.5264	.4892	.4567	.4280	.4026
20	.2045	1.001	2.013	2.938	3.702	5.737	6.348	6.443	6.291	6.123	5.885	5.638	5.395	5.163	4.943	4.738
30	.1058	.6931	1.833	3.406	5.139	12.58	16.63	18.61	19.49	19.76	19.69	19.42	19.04	18.60	18.13	17.64
40	.06094	.4341	1.285	2.640	4.432	16.07	26.07	32.74	36.87	39.34	40.72	41.37	41.55	41.41	41.05	40.54
50	.03854	.2842	.8777	1.893	3.345	15.54	30.17	42.64	42.68	58.73	63.38	66.54	68.59	69.82	70.46	70.66
60	.02594	.1949	.6152	1.359	2.468	13.26	29.36	45.96	60.51	72.34	81.58	88.63	93.92	97.80	100.6	102.5
70	.01820	.1383	.4424	.9920	1.830	10.75	26.04	44.12	62.00	78.10	92.23	103.3	112.5	119.9	125.8	130.4
80	.01311	.1004	.3242	.7341	1.368	8.519	21.98	39.49	58.47	77.05	94.11	109.2	122.2	133.2	142.5	150.1
90	$.0^{2}9594$	.07397	.2404	.5481	1.029	6.677	18.01	33.83	52.21	71.37	90.04	107.4	123.2	137.1	149.4	160.0
	VALUES OF v AT ABUTMENT															
10	-995.6	-990.5	-986.9	-984.3	-982.3	-976.4	-973.4	-972.0	-970.9	-970.1	-969.5	-969.1	-968.7	-968.4	-968.2	-968.0
20	-998.2	-990.9	-981.3	-971.9	-963.7	-936.3	-921.3	-911.8	-905.4	-900.4	-896.7	-893.7	-891.3	-889.3	-887.7	-886.2
30	-999.4	-995.9	-989.0	-979.3	-968.2	-915.0	-878.1	-853.0	-835.0	-821.5	-811.0	-802.5	-795.6	-789.8	-784.8	-780.6
40	-999.7	-998.1	-994.4	-988.4	-980.2	-923.1	-867.0	-822.8	-788.7	-762.2	-741.1	-723.9	-709.6	-697.6	-687.3	-678.5
50	-999.9	-999.1	-997.0	-993.5	-988.4	-943.2	-883.8	-827.3	-818.8	-739.0	-705.9	-678.0	-654.6	-634.6	-617.3	-602.3
60	-999.9	-999.5	-998.3	-996.3	-993.1	-961.3	-910.5	-853.7	-799.3	-750.4	-707.6	-670.5	-638.3	-610.2	-585.7	-564.0
70	-1000.	-999.7	-998.5	-997.7	-995.8	-974.3	-935.3	-886.1	-834.1	-783.6	-735.6	-694.2	-656.0	-622.0	-591.5	-564.3
80	-1000.	-999.8	-999.4	-998.6	-997.4	-983.0	-954.7	-915.8	-871.2	-824.9	-779.5	-736.5	-696.5	-659.8	-626.2	-595.6
90	-1000.	-999.9	-999.6	-999.1	-998.3	-988.8	-968.8	-939.7	-904.2	-865.3	-825.3	-785.8	-747.8	-712.0	-678.4	-647.2

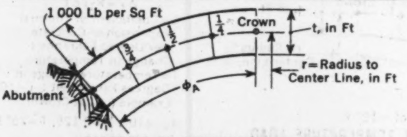


DIAGRAM OF CONCENTRATED RADIAL LOAD AT ABUTMENT

Thrust, Moment, Shear,
in Lb, and Ft-Lb

$H = h$

$M = m r$

$V = v$

TABLE 9(*b*).—RADIAL DEFLECTIONS FOR CONCENTRATED RADIAL LOAD AT ABUTMENT

ϕ_A	VALUES OF t/r															
	.025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	.1000	1.125	1.250	1.375	1.500
	VALUES OF K_1 AT CROWN															
10	-808.3	-1434	-1592	-1653	-1633	-1726	-1736	-1740	-1742	-1744	-1745	-1745	-1746	-1746	-1746	-1747
20	1053	258.4	-344.8	-730.7	-974.1	-1412	-1518	-1561	-1583	-1596	-1605	-1611	-1616	-1619	-1622	-1624
30	1415	1125	708.4	442.5	143.2	-726.3	-1050	-1195	-1272	-1318	-1349	-1370	-1386	-1398	-1407	-1415
40	1501	1376	1217	1038	851.5	55.45	-410.9	-670.5	-823.2	-919.9	-965.1	-1031	-1066	-1092	-1113	-1129
50	1537	1469	1385	1288	1182	454.1	191.5	-112.6	-466.4	-459.8	-558.1	-631.2	-686.6	-729.7	-763.9	-791.7
60	1561	1517	1461	1407	1343	984.4	642.0	364.4	152.6	-65.01	-126.7	-218.8	-290.8	-347.9	-394.1	-432.1
70	1581	1550	1514	1475	1433	1194	946.0	740.5	535.4	383.3	264.8	163.3	83.97	19.18	-34.32	-78.99
80	1601	1576	1550	1522	1449	1326	1170	978.4	826.2	695.5	584.8	491.7	414.7	349.5	294.5	247.7
90	1621	1674	1581	1559	1537	1417	1288	1161	1043	937.2	844.2	763.6	694.0	633.6	582.4	537.7
	VALUES OF K_1 AT $\frac{1}{4}$ POINT															
10	-728.0	-1466	-1608	-1662	-1689	-1727	-1736	-1740	-1742	-1743	-1744	-1744	-1745	-1745	-1745	-1746
20	728.0	38.14	-490.2	-833.0	-1045	-1433	-1528	-1566	-1586	-1597	-1605	-1610	-1614	-1617	-1620	-1622
30	1039	796.3	459.6	210.8	-48.64	-809.3	-1094	-1221	-1289	-1330	-1357	-1375	-1389	-1399	-1407	-1414
40	1108	1009	878.5	728.7	571.5	-109.9	-514.0	-739.0	-871.8	-955.8	-1012	-1053	-1082	-1104	-1122	-1136
50	1089	1083	1017	939.4	853.4	395.8	22.35	-236.6	-591.9	-531.6	-617.0	-679.2	-726.3	-762.8	-791.7	-815.1
60	1150	1118	1079	1034	984.6	696.2	414.1	183.4	66.01	-126.3	-226.7	-303.5	-363.6	-411.0	-449.3	-480.6
70	1162	1140	1112	1086	1052	868.7	671.1	490.7	338.6	214.8	120.3	35.75	-28.63	-81.10	-124.3	-160.3
80	1172	1156	1138	1118	1096	972.3	835.6	703.0	583.3	480.1	392.8	382.3	258.5	207.9	161.8	127.7
90	1183	1170	1156	1141	1125	1037	931.8	826.4	721.8	629.7	546.0	524.1	493.2	447.2	408.8	373.9
	VALUES OF K_1 AT $\frac{1}{2}$ POINT															
10	-1192	-1556	-1650	-1687	-1705	-1731	-1737	-1739	-1740	-1741	-1741	-1741	-1742	-1742	-1742	-1742
20	-128.7	-547.1	-880.1	-1098	-1238	-1494	-1557	-1581	-1593	-1600	-1605	-1608	-1610	-1612	-1613	-1614
30	535.1	-70.71	-256.6	-407.3	-563.5	-1035	-1215	-1296	-1338	-1363	-1379	-1390	-1397	-1403	-1408	-1411
40	838.9	467.8	-13.45	-88.94	-172.3	-557.6	-795.8	-929.4	-1010	-1057	-1089	-1111	-1128	-1141	-1149	-1156
50	883.2	770.5	550.4	243.2	-67.01	-381.1	-436.3	-576.6	-641.5	-736.0	-781.4	-814.3	-838.9	-855.6	-869.6	-880.6
60	860.5	835.8	759.5	639.4	476.9	-68.13	-197.0	-306.7	-391.6	-455.2	-502.5	-538.1	-565.2	-586.1	-602.5	-615.5
70	812.0	821.0	803.0	760.7	696.8	144.1	-58.66	-130.2	-191.8	-241.9	-276.9	-312.9	-337.4	-356.7	-372.1	-384.4
80	751.0	772.1	778.9	772.3	753.6	680.1	147.1	-25.83	-35.79	-96.82	-124.3	-208.5	-243.4	-271.1	-295.6	-319.0
90	747.7	704.3	719.2	725.0	721.6	580.3	258.2	-11.79	-54.59	-96.56	-93.49	-74.73	-24.60	-52.58	-58.13	-62.74
	VALUES OF K_1 AT $\frac{3}{4}$ POINT															
10	-1539	-1672	-1708	-1722	-1729	-1737	-1737	-1737	-1737	-1737	-1737	-1737	-1736	-1736	-1736	-1736
20	-1177	-1273	-1376	-1448	-1495	-1580	-1598	-1603	-1604	-1604	-1603	-1605	-1602	-1602	-1601	-1636
30	-1117	-1121	-1154	-1184	-1221	-1344	-1388	-1404	-1410	-1412	-1411	-1410	-1409	-1408	-1406	-1405
40	-1121	-1102	-1093	-1092	-1097	-1146	-1181	-1197	-1202	-1202	-1201	-1197	-1194	-1191	-1187	-1184
50	-1131	-1109	-1091	-1078	-1068	-1048	-1043	-1039	-1160	-845.0	-1015	-1006	-997.0	-988.9	-981.4	-974.5
60	-1141	-1121	-1103	-1087	-1073	-1019	-981.2	-953.5	-927.8	-904.7	-883.9	-865.1	-848.2	-832.9	-819.1	-806.5
70	-1153	-1135	-1119	-1103	-1089	-1026	-979.2	-927.9	-886.7	-849.8	-813.2	-785.4	-760.6	-736.8	-715.4	-696.0
80	-1162	-1147	-1133	-1119	-1106	-1044	-988.3	-917.3	-886.7	-841.2	-799.4	-761.4	-726.8	-695.4	-667.0	-641.1
90	-1174	-1161	-1149	-1137	-1126	-1080	-1045	-1019	-999.2	-982.7	-968.0	-830.1	-737.9	-702.0	-668.3	-637.7
	VALUES OF K_1 AT ABUTMENT															
10	-1777	-1768	-1762	-1757	-1753	-1743	-1738	-1735	-1733	-1732	-1731	-1730	-1729	-1729	-1728	-1728
20	-1782	-1769	-1752	-1735	-1720	-1671	-1645	-1628	-1616	-1607	-1600	-1595	-1591	-1587	-1584	-1582
30	-1784	-1778	-1765	-1748	-1728	-1633	-1567	-1523	-1491	-1466	-1448	-1432	-1420	-1410	-1401	-1393
40	-1785	-1782	-1775	-1764	-1750	-1648	-1548	-1469	-1408	-1361	-1323	-1292	-1267	-1245	-1227	-1211
50	-1785	-1783	-1780	-1773	-1764	-1684	-1578	-1477	-1462	-1319	-1260	-1210	-1168	-1133	-1102	-1075
60	-1785	-1784	-1782	-1778	-1773	-1716	-1625	-1524	-1427	-1339	-1263	-1358	-1139	-1089	-1045	-1007
70	-1785	-1784	-1783	-1781	-1778	-1739	-1669	-1648	-1489	-1399	-1313	-1239	-1171	-1110	-1056	-1007
80	-1785	-1785	-1784	-1782	-1780	-1755	-1704	-1635	-1555	-1472	-1391	-1315	-1243	-1178	-1118	-1063
90	-1785	-1785	-1785	-1784	-1784	-1777	-1762	-1738	-1707	-1672	-1634	1460	-1335	-1271	-1211	-1155

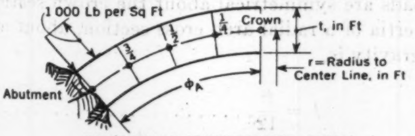


DIAGRAM OF CONCENTRATED RADIAL LOAD AT ABUTMENT

Radial Deflection, in Ft

$$\Delta_r = \frac{K_1}{E_c}$$

E_c = Modulus of Elasticity of Concrete in Direct Stress, Lb per Sq Ft

GENERAL ASSUMPTIONS MADE IN ANALYZING ARCH ELEMENTS

In their paper in this Symposium, Messrs. Houk and Keener present a thoroughly inclusive list of assumptions that affect the design of masonry dams. Those that apply to circular arch dams of uniform depth are as follows:

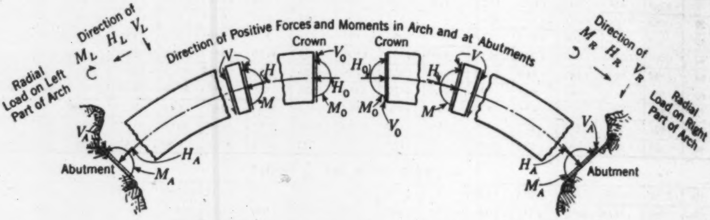

Direction of
M_L H_L V_L
Radial Load on Left Part of Arch
Direction of Positive Forces and Moments in Arch and at Abutments
Crown
Crown
V
H
M
V_0
H_0
M_0
V_A
H_A
M_A
Abutment
Direction of
M_R H_R V_R
Radial Load on Right Part of Arch

FIG. 2.—DIRECTION OF FORCES AND MOMENTS

(*a*) The arch material is homogeneous and isotropic;

(*b*) Hooke's law applies, and the proportional elastic limit is not exceeded;

(*c*) A plane section before bending remains plane after bending;

(*d*) Direct stresses have a linear variation from extrados to intrados;

(*e*) The modulus of elasticity in direct stress is the same for tension and compression;

(*f*) Temperature strains are proportional to temperature changes;

(*g*) The temperature change occurs uniformly throughout an arch;

(*h*) The abutments of arch elements are radial planes;

(*i*) The ratio of the modulus of elasticity in direct stress (E) to the shearing modulus of elasticity (G) is expressed by

$$\frac{E}{G} = 2\,(1 + \mu) \qquad (2)$$

in which μ = Poisson's ratio;

(*j*) The symbol K expresses the ratio of the detrusion caused by the actual shear distribution to that caused by an equivalent shear distributed uniformly. Assuming that $K = 1.25$ and $\mu = 0.2$, the ratio $\frac{K}{G} = \frac{1.25 \times 2.4}{E} = \frac{3}{E}$, which is the ratio included in Tables 3 to 9 (Appendix).

Further Assumptions Applying to the Tables.—In Tables 3 to 9, furthermore, it is assumed that:

(*k*) The arch is circular and of uniform thickness (that is, t and r are held constant), and 1 ft wide;

(*l*) The arch and its loads are symmetrical about the crown section;

(*m*) The moment of inertia of a radial arch cross section, about a vertical line through its center of gravity is,

$$I = \frac{t^3}{12} \qquad (3)$$

NOTATION AND UNITS

The notation and units, and the arguments, required to enter Tables 3 to 9 are given on each table. All dimensions are in units of feet; and forces and moments are in units of pounds and pound-feet. The process described in this paper is for a modified radial adjustment. For the amplified method the reader is referred to "Trial Load Method of Analyzing Arch Dams."[5]

PREPARATION OF FOUNDATIONS

By Charles H. Paul,[7] and Joseph Jacobs,[8] Members, Am. Soc. C. E.

Synopsis

It is assumed that only rock foundations are considered in this paper, except as indicated in the section headed "Special Conditions." The term "foundation" is taken to mean not only the entire area on which the dam rests, but also the areas upstream and downstream and out into the abutments adjacent to the limits of the superimposed structure, as far as the loading, seepage, or scour criteria may require.

Let it be stated at the beginning that no amount of text or treatise material can take the place of sound, experienced judgment in passing on conditions and requirements for dam foundations. All that the writers can hope to accomplish is to emphasize the more important requirements and to discuss, briefly, some general methods of treatment. Every foundation for an important dam is a problem in itself and should be given attention by men of broad experience and sound judgment in such matters.

Main Considerations

Assuming that preliminary investigations have indicated that a suitable foundation is available, the main considerations for the preparation of such foundation are: (1) Bearing capacity, (2) bond (with masonry), (3) sliding or frictional resistance, (4) control or reduction of uplift pressure, (5) prevention or control of seepage, and (6) prevention or control of scour from overflow or outlets. The general nature of these considerations is defined in this section, with methods of treatment presented in somewhat greater detail in subsequent sections.

Bearing Capacity.—For want of a better term, "bearing capacity" is used to indicate the supporting characteristics of the rock over large areas. In that respect it is synonymous with compressive strength; but the latter term usually applies to the strength of rock fragments or blocks, and there are numerous instances of hard rock so interspersed with seams that the blocks may readjust themselves under heavy loading and thus result in settlement or movement that would not be permissible in a dam foundation. Basaltic rocks are often in this condition, and decay in badly weathered rock sometimes follows horizontal and vertical seams, leaving blocks of sound, hard rock between. The important requirements for bearing capacity are not only the compressive strength of the rock itself, but also its adequate bearing strength over the entire area of the foundation.

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[8] Cons. Engr., Seattle, Wash.

In general, most firm, non-decomposed rocks have sufficient bearing capacity for dam foundations, but special consideration is necessary for high dams. Seamy, fissured, faulted, and stratified rocks present special problems which must be dealt with in accordance with the loading and other conditions. Some of the approved methods for treating such foundations are discussed subsequently under other headings. Cavernous rocks present so many special problems that they are dealt with (although necessarily in a brief manner) under the heading "Special Conditions." It is sometimes necessary, and practicable, by washing out and grouting seams, and by "dental work," to reinforce or solidify a foundation that is quite unsatisfactory in its natural state, so as to make it entirely adequate for the purpose desired. Such work may have to be extended over the entire area and into the subfoundation to considerable depth in order to satisfy the requirements fully. Obviously, care and good judgment are essential in its execution.

Bond.—It now is generally accepted as a fact that by the use of approved methods a tight bond can be secured at the contact between clean, hard rock and the masonry of a dam. To accomplish this, however, the rock surface must be absolutely clean. A grout wash or mortar coat, preferably spread with wire brushes, immediately preceding the placing of the masonry, gives added assurance of accomplishing the desired result. A thin film of dirt, oil, or grease, or other similarly objectionable foreign matter, may weaken or destroy what otherwise should result in a tight, firm bond capable of meeting all requirements for contact between the dam and its foundations.

Fragile, crumbly rocks present special problems, although their bearing capacity may be satisfactory. Shales also require special study and treatment as to bonding methods. In cases where the character of the foundation is such that a firm, tight bond cannot be assured, then keying into the foundation, and other suitable modifications of the design of the dam, may be sufficient to secure safe construction.

Sliding or Frictional Resistance.—On firm, hard rock, where a tight bond can be secured, there is no danger of sliding at the contact surface. In such cases, movement could occur only by shearing. However, in stratified rocks, where clay or soft seams of appreciable thickness occur below the prepared surface, the danger of sliding in the foundation itself must be taken into consideration. Cohesion and friction factors in such cases must be studied carefully. Washing out and grouting the seams may be practicable. Often it is necessary to set the dam deep into the foundation to secure a downstream shoulder that will serve as a buttress to prevent movement in that direction.

The precautions mentioned under the preceding subheading, "Bond," should also be given full consideration under this subheading in all cases where a tight bond cannot be secured between the dam and the foundation rock.

Control or Reduction of Uplift Pressures.—Grouting foundation seams, and the combination of a grout curtain and drainage, are the measures usually relied upon to control or reduce uplift pressure. The extent to which uplift pressure and its distribution should be provided for in the final design will depend on the character of the foundation rock, the probable success of the grouting operations, and the probable effectiveness of the drainage system.

Prevention or Control of Seepage.—Measures taken to control uplift pressures work automatically (as far as they go) to help prevent or control seepage. Seepage control, however, may require much more extensive work. Seepage may find its way not only under the dam, but also out around or through the abutments. Upstream earth blankets or concrete aprons, washing and grouting seams, tunneling out and plugging large seams, cutoffs or grout curtains in foundation and abutments and out beyond the limits of the structure, all have their place under certain conditions. Rock blankets on downstream banks, or other forms of downstream drainage, may be required to control such seepage as cannot assuredly be prevented.

Protection Against Scour.—Protection against scour is a subject of great importance and involves many considerations. The hydraulic-jump principle, in the design of protection features, is the basis of the more common methods of treatment. Where a deep tailwater is assured, very little may be required in the way of special foundation treatment; but in many cases a properly designed apron is necessary, preferably designed by use of the hydraulic-jump formula, supplemented by model tests. In other cases, as at Grand Coulee Dam, in Washington, a specially designed bucket is effective.

Even so-called solid rock usually has some seams into which water can penetrate. Water in such seams acts effectively as a wedge; and the pounding of falling water acts as a mighty impact force on these water wedges and tends to disrupt the rock unless it is properly protected. If the tailwater depth is not sufficient protection, then the need for a concrete apron is indicated. Sometimes, when conditions are not severe, a comparatively thin concrete apron will blanket the rock seams and thus serve as sufficient protection. All such aprons should be so designed as to safely resist possible uplift pressures.

INVESTIGATIONS

The importance of thorough preliminary surface and subsurface explorations is obvious. As an aid to sound conclusions, a competent engineering geologist should have continuous contact with all such explorations, working in cooperation with the engineers. This will make available an intelligent interpretation of the geologic history, and of the origin, structure, and character of the rock, as they apply to the engineering requirements.

These preliminary investigations, however, are only a first step in the examination and study of the foundation. The observations and studies should be continued after the rock has been exposed by removal of overburden, and should include detailed surface and subsurface examinations up to the time of its final preparation. Even in the best foundations there are likely to be weak spots, fractured areas, slips, or seams, which require special study after full exposure and after all pertinent information is available as obtained by stripping, core drilling, open shafts, drifts, and other means.

More detailed exploration of subsurface conditions should follow the preliminary investigations of topography, geology, and preliminary borings, at sites where these early studies indicate favorable conditions. These more detailed explorations usually take the form of test pits, trenches, abutment drifts, and core drilling with suitable types of drilling machines.

Exploratory Drilling.—Various types of core drills are in use—the diamond drill, the shot drill, and some of the newer devices with cutting points of special alloy steels for use, mainly, in sedimentary rocks. The diamond drill is best for use in the harder rocks and is probably a little faster than the shot drill. It may be noted, too, that with the latter the shot is sometimes lost in considerable amounts in large open seams or crevices in the rock. There is now available an excellent light-weight portable diamond drill suitable for exploration in rough country, or in galleries or other places where space is limited. These little rigs require no tripod or other mounting except direct clamping to the casing pipe in the hole to be drilled. Good equipment and good operating technique are essential for best results. The more usual practice is to drill vertical holes; but inclined holes are entirely practicable and are frequently used in foundation exploratory work.

Although inclined holes are somewhat more difficult to drive, and cost a little more per linear foot than vertical holes, they are effective, particularly where a zone of some width is to be explored. For this reason, they are sometimes more desirable and more economical than the greater number of vertical holes that would be required to develop the same information. In bad foundations requiring extra precaution, or where vertical or nearly vertical seams are anticipated, a high percentage of exploratory coverage may be obtained by a drilling pattern which combines vertical holes and inclined holes, alternately.

The required number and depth of drill holes will depend mainly upon the local rock structure and upon the magnitude and importance of the proposed dam. They should be adequate, however, to reveal, definitely, the character of the foundation rock. They can generally be driven, at moderate cost, to practically any depth, and at practically any angle that reasonably may be required. The core should be labeled and indexed methodically, for both immediate and subsequent critical examination. All nonrecovery sections of the core should have dummy fillers so marked as to indicate the probable reason for the nonrecovery. As an aid to correct core interpretation, laboratory tests of the rock, as to its compressive strength, modulus of elasticity, hardness, soundness, solubility, permeability, and absorptive properties, are desirable.

In general, crystalline rocks, such as granite, and thoroughly indurated sedimentary rocks, such as quartzite, yield the best cores. The softer sedimentary rocks, such as loosely bound sandstone and shales, yield the poorer cores. A poor core recovery may mean a poor rock throughout; it may mean the piercing of extensive crush zones and seams in otherwise good rock; and it may mean too small a drill hole, or poor equipment, or poor technique in operation. Cores of less than about 1-in. diameter should generally be avoided because of their greater breakage tendency and their greater susceptibility to failure under inexpert drill operation. Diamond-drill cores of a diameter from $1\frac{1}{2}$ in. to 2 in. are the more desirable, and shot-drill cores, for best results, should generally be from 3 in. to 5 in. in diameter.

In drilling operations, the ground-water table should always be located, if possible, its elevation carefully noted, and its fluctuation during progress of the work recorded and studied. Similarly, any water flows or seepage encountered in drilling should be noted as to position and volume, for such data serve as partial indexes of rock tightness. The major facts to be observed, and to have in mind, in the process of core drilling, may be listed as follows: (*a*) Depth of overburden; (*b*) depth to ground-water table; (*c*) depths at which drilling water escapes; (*d*) character and classification of the rock; (*e*) condition and quality of the rock; (*f*) evidences of processes affecting the rock, such as crushing, weathering or other forms of decay, and solution effects; (*g*) nonrecovery of core and the reasons therefor; (*h*) porosity and permeability of the rock, and seepage conditions in the rock structure as a whole; (*i*) any required tests to show depths and rates of leakage into hole from ground water; and (*j*) any required tests to show rate of leakage out of hole under drilling pressure or higher testing pressure.

Core Drilling Interpretations.—The purpose of core drilling is to secure, as nearly as possible, undisturbed samples, to predetermined depths, of the foundation rock that is to support the dam. Usually such drilling, in the aggregate, involves considerable expense. Frequently the cores may be impaired by undue breakage, or by undue grinding into granular particles, due to the use of poor or unsuitable drilling equipment or poor technique in operating the equipment. Such cores may easily be misinterpreted. Too often the interpretation of drill cores is treated as if it were of only casual importance, whereas much depends upon correct interpretation. It is as important and necessary as the securement of the cores themselves, and is the only justification for the considerable expense involved.

It is very desirable to have the drilling supervised continuously by an experienced man with a knowledge of engineering geology and also of the technique of drilling and of core recovery. This provides for intelligent interpretation and control as the work proceeds, including modification of the drilling program where advisable. It also affords opportunity to determine promptly the reasons for nonrecovery of core when such instances occur and, if equipment or technique rather than rock structure is responsible therefor, to apply the necessary remedies.

The percentage of core recovery, of course, is something of an index of rock character but by no means an infallible index. A good core recovery is a favorable indication, but a relatively poor recovery, for reasons mentioned in a previous paragraph, is not necessarily an indication of bad rock. From all of the foregoing it is clearly evident that core drilling calls for expert interpretation, and it should be appreciated that such interpretation requires a background of thorough knowledge and understanding of the engineering geology of the dam site. Without correct interpretation of results, it is obvious that the potential value of core drilling work is largely lost. Furthermore, incorrect or incomplete interpretation may lead to incorrect treatment of the foundation and even to incorrect design of the dam.

Large-Size Shot Drills.—The large-size shot drill is a piece of equipment that has become almost indispensable in the examination of important foundations.[9] By its use, a good check may be had on the interpretations of the small drill cores. This drill leaves a clean hole, 36 in. to 40 in. in diameter according to the size of the drill bit selected, with the rock walls undisturbed and clearly exposed in their natural condition. A true and reliable picture is given, not only of the rock in place, but also of the size, character, and inclination of seams. Pockets or caverns, if intersected, are sliced through cleanly without disturbance, and are thus made accessible for further examination. It has been demonstrated that a 36-in. or 40-in. hole can be drilled to a depth of 50 to 60 ft without great difficulty. It is practicable to go to depths of 75 to 100 ft if necessary or particularly desirable. An observer may descend into these holes, by ladder or by sling, and examine the walls at leisure, and the results of such examinations are not confused by suspicions of artificial disturbances, as is often the case when shafts are driven by ordinary rock excavation methods, even where only light blasting is permitted.

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[9] *Proceedings*, Am. Soc. C. E., June, 1939, p. 953.

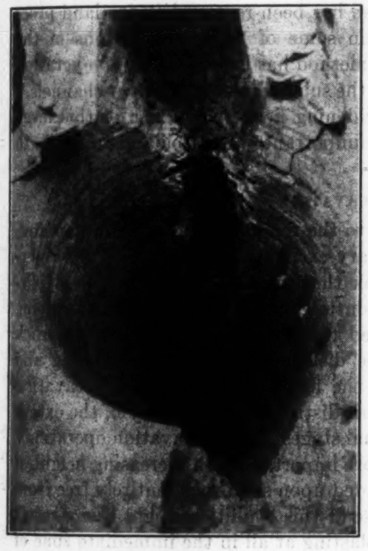

FIG. 3.—VIEW OF LARGE CORE HOLE, TENNESSEE VALLEY AUTHORITY

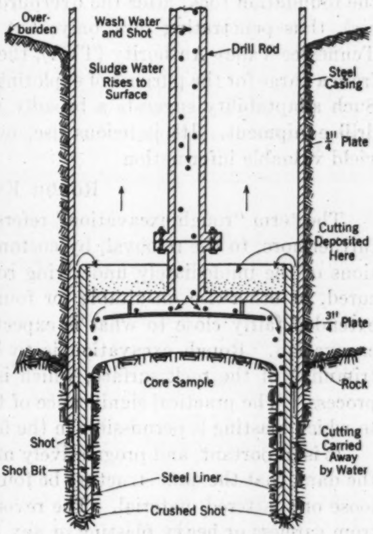
Over-burden
Wash Water and Shot
Drill Rod
Sludge Water Rises to Surface
Steel Casing
¼" Plate
Cutting Deposited Here
¾" Plate
Core Sample
Rock
Cutting Carried Away by Water
Shot
Shot Bit
Steel Liner
Crushed Shot

FIG. 4.—SKETCH SHOWING ACTION OF LARGE-DIAMETER CORE DRILL

Fig. 3 shows more clearly than can be expressed in words the advantage of these large holes as a means of subsurface investigation. It is of particular value in that it shows in one view the clean exposure of the rock, as well as the exposure of a cavernous solution channel and the convenient access to it.

Fig. 4 shows the method of operating this drill. The unit cost of drilling holes with this equipment varies considerably, depending upon the character of the rock, depth of hole, and the difficulty of removing the core; in rough figures it may be given as from $10 to $30 per ft. The character of the rock, of course, affects the speed of drilling, but another significant cost item is the removal of the core. One method of removing core is to drop a very small charge of powder down along one side of the core and crack it off in 2 to 3 ft lengths. Often the core can be cracked off by wedging. If the core cracks through without fracturing otherwise, it is easily removed by means of a lifting device inserted into a small hole drilled down the core axis. In case the core breaks up into small pieces, the job of mucking out is slow, and becomes an important factor in the cost. In such cases it usually is economical to use the 40-in. drill, so that the workman is not so cramped for room in cleaning out the hole.

In ordinary hard rock, the core from these large-size holes may be removed in sections, properly matched and oriented, and laid out on the ground as an exhibit of the character and structure of the foundation rock; but by far the greatest value of this type of exploration lies in the opportunity to examine the rock in place exactly as Nature presents it for the use of Man.

The most common method of using these large drills is to set them up on the foundation rock, after the overburden has been removed (a dry-land process), thus penetrating rock only; but on some of the investigations of the Tennessee Valley Authority (TVA), the method has been adapted to operation from a barge for the purpose of exploring the submerged rock in river channels.[10] Such adaptability suggests a broadly widening field for the use of this large drill equipment. Its judicious use, on important jobs, is almost certain to yield valuable information.

Rough Excavation

The term "rough excavation" refers to the removal of all overburden and, furthermore, to the removal, by customary excavation methods, of those portions of the immediately underlying rock that are soft, weathered, badly fissured, or otherwise unsuitable for foundation purposes, this operation being extended fairly close to what is expected to be the final surface of the rock excavation. Rough excavation is to be distinguished from the scaling and trimming of the rock surface, which is the final step in the rock excavation process. The practical significance of this distinction lies mainly in the extent to which blasting is permissible in the final stages of the excavation operations.

It is important, and progressively more important with increasing height of the dam, that the dam structure be founded upon solid rock, entirely free from loose or shattered material. The reverse of this condition might easily result from careless or heavy blasting or any blasting at all in the immediate zone of final excavation. It is necessary, therefore (and customary), to limit the extent of permissible blasting and to define clearly such limitation in the specifications. A not unusual practice is progressive blasting limited to depths of about two thirds the total estimated remaining depth of excavation, with decreasing charges of explosives for the shallower depths, until within a foot or two of the foundation level, when blasting is prohibited and the remaining material is removed by barring, wedging, use of a paving breaker, or other means not requiring blasting.

An excellent example of a provision for control of blasting is that found in the specifications for construction of Grand Coulee Dam, which are as follows:

> "*Rock Excavation for Foundations for Dam and Power House.*—The excavation for the dam and power house shall be made to sufficient depth

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[10] "Underwater Exploration with Calyx Drills," by Hendon R. Johnston, *Engineering News-Record*, March 24, 1938, p. 436.

> to secure foundation on sound ledge rock, free from weathered material, open seams, or other objectionable defects, as determined by the contracting officer. All necessary precautions shall be taken to preserve the rock below and beyond the lines of excavation in the soundest possible condition. All blasting operations, the depth and size of holes, and the size and characteristics of the charge shall be subject to the approval of the contracting officer. The explosives shall be of such quantity and moderate power and used in such locations as will neither open up seams nor crack or damage the rock outside the prescribed limits of excavation. The firing of systems of blasts shall be controlled by the use of delay exploders. No hole for blasting shall be drilled more than two-thirds the depth of the proposed excavation. Where the depth of excavation is less than 15 feet, the holes shall not be larger than necessary to permit the passage of whole sticks of powder to the bottom of the hole and such holes shall not be sprung or chambered. As an excavation approaches its final lines the depth of holes for blasting and the amount of explosives used per hole shall be progressively reduced. Whenever, in the opinion of the contracting officer, further blasting is liable to injure the rock upon or against which concrete is to be placed, the use of explosives shall be discontinued and the excavation completed by wedging, barring, channeling, line drilling and broaching, or other suitable methods. The excavation for the base of the dam and power house at all elevations shall be so shaped and roughly stepped where necessary, as determined by the contracting officer, to produce the desired surface of contact between the concrete and the foundation rock."

A good practice in the excavation of dam foundations is to require that the overburden be removed sufficiently to permit an early inspection of the rock before its actual excavation is begun. This is desirable in order that a more intelligent program, and a more detailed procedure, as to rock excavation and surface treatment of the rock, may be outlined in advance. In some cases, it may be desirable to explore the rock further with jackhammer or with large-diameter drills. A program thus outlined may need to be revised, upon full exposure of the rock, but that possibility does not at all lessen the desirability of securing the advance information suggested herein. Such advance information is always valuable.

Depending upon the character of the rock, the extent of weathering that has occurred, the steepness of the abutments, etc., the relative amounts of rough excavation, and of scaling and trimming, on different jobs, are widely variable. On Boulder Dam, for example, these relative amounts were 68% and 32%, respectively, of the total rock removed, and similar data for the Owyhee Dam, in Oregon, were 85% and 15%, respectively. On the Arrowrock Dam, in Idaho, after removal of the overburden, the volume of rock excavation required in the river-channel section was negligible and only on the abutments was there need for appreciable quantities of rough rock excavation or of scaling. Although rough excavation almost always constitutes the major part of the total foundation excavation, it is apparent from the foregoing that scaling and trimming may become a significant item of cost in some cases.

Final Excavation and Finish

"Scaling and trimming" is the final removal of slabby or "drummy" rock, including all pieces partly loosened by the work of rough excavation or partly separated from the main rock mass by seams or cracks. Should any weathered or partly decomposed rock remain after the rough excavation is finished, it also should be removed during the scaling and trimming operations. All this work should be done with great care so as not to leave on the foundation surface any rock other than that which is an integral part of the rock mass. It is customary, in this operation, to prohibit or strictly limit the use of powder, requiring the scaling and trimming to be done with picks, paving breakers, bars, and wedges.

Areas of low bearing capacity, steeply inclined seams, faults, or crush zones, in an otherwise good foundation, always call for special consideration. Their relative importance depends on their size or area; whether they are, or can be, fully confined; whether they can be protected against further deterioration; and whether their bearing capacity, even though below the average of the foundation, is still sufficient to carry the load without undue yielding. Usually these doubtful areas are cleaned out to sufficient depth and refilled or plugged with concrete, in which cases their size, shape, and condition control the depth to which excavation should go. It is often desirable that this cleaning out and plugging be followed by grouting as an additional precaution, particularly near the upstream face of the dam where seepage control is important.

It is often assumed that relatively small areas, whether treated or not, that might not carry their share of the load will be spanned by the masonry of the dam and thus be rendered harmless. Often this may be true, but any such condition means some concentration of stresses in the masonry which should not be overlooked. It may be negligible, or it may be important, depending upon the extent and character of the unbalanced distribution of loading. In general, it may be stated that, in an otherwise good foundation, areas of low bearing capacity are troublesome, but if given proper treatment they usually are not serious.

The prepared foundation surface should be free from sharp angles of such extent as might tend to induce shrinkage or shearing stresses in the concrete of the dam. Top edges of benches, or the like, should be chamfered approximately 45° so as not to be objectionable in this respect. Pinnacles or sharp projections should be knocked off, and prominent knobs should be flattened off for the same reason. Deep potholes and narrow gorges should be filled with concrete, so that it can set and cool and take its initial shrinkage before the concrete of the dam is placed. Any abrupt changes in elevation of the foundation tend to induce shearing stresses in the concrete of the dam because of unequal shrinkage. Corrective measures may be applied in preparing the foundation and in placing the concrete in short lifts against the rock face so as to permit initial shrinkage to take place in the lower lifts before the higher ones are placed. Fig. 5 shows a part of the foundation of Grand Coulee Dam, scaled and trimmed and ready for final cleaning.

Following the scaling and trimming, the foundation surface should be thoroughly cleaned of all loose particles, including the very finest of the chips, sand, and dirt. Immediately in advance of placing concrete, the foundation should be scrupulously clean. To accomplish this it is a common practice to use a strong air and water jet, powerful enough to dislodge any particles loosely attached to the rock surface. The dirty water that accumulates in puddles is removed in buckets or by other convenient methods. The washing and scrubbing process should be continued until the water in these puddles is clear, and entirely free from dirt. In the final cleaning process, sponges are commonly used to sop up the last of the wash water. Too much emphasis cannot be given to the importance of having an absolutely clean foundation on which to place the concrete of the dam. The slightest film of dirt, oil, or grease may prevent a tight bond, and to a large extent defeat the purpose of the vigilant supervision and the expensive work involved in foundation preparation.

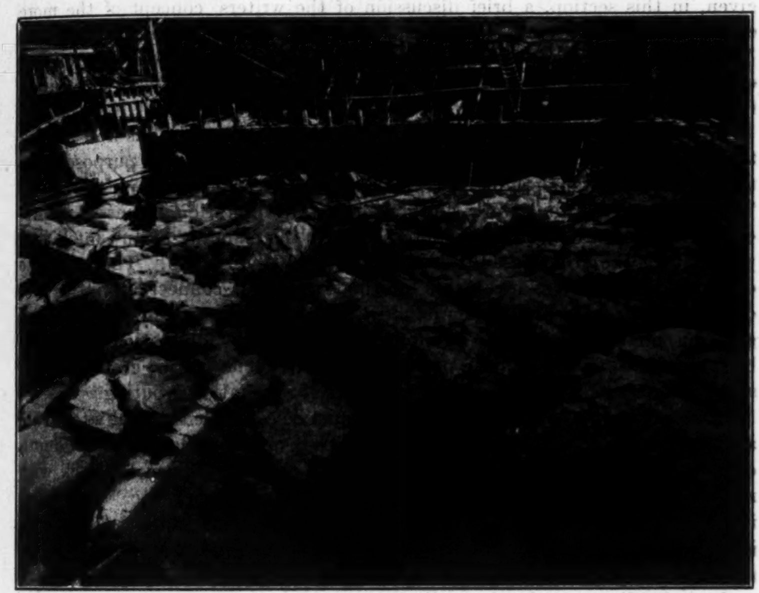

FIG. 5.—VIEW OF THE FOUNDATION OF GRAND COULEE DAM, READY FOR FINAL CLEANING

Where well-defined cutoffs are required, or in other places where it is necessary to adhere strictly to prescribed lines, line drilling (or channeling and broaching in shale or the softer rocks) is often resorted to. This is expensive work and should be required only where adherence to prescribed lines is essential. Frequently, in excavation for cutoffs, for example, it may be practicable to allow some tolerance for overbreak. On the other hand, in seamy rocks where the effect of blasting is likely to extend beyond the prescribed lines, these more expensive methods are justified on the basis of additional safety and economy. The faces of downstream shoulders, where required to furnish resistance against sliding, are often best prepared by means of line drilling.

Grouting and Drainage

This paper would be strikingly incomplete without some reference to grouting of foundations. Therefore, although James B. Hays, M. Am. Soc. C. E., has presented a complete paper that deals with the general subject,[106] there is given, in this section, a brief discussion of the writers' concept of the more essential features of cement grouting as applied to dam foundation preparation. No comments are made on asphalt grouting, clay grouting, chemical grouting, or other forms, for the reason that they are not commonly used in treating foundations for masonry dams.

Although of relatively modern application, cement grouting is now a practically universal procedure in masonry-dam foundation work, its purpose being to consolidate the underlying rock, and to reduce seepage and uplift pressure. The exploratory drilling work, and its correct interpretation, discussed in preceding sections, are indispensable precedents to the planning of any intelligent program for the grouting and drainage of a dam foundation. The large-size drill hole also provides invaluable information in this connection. It is also desirable that some preliminary studies be made to determine the range of grouting pressures to which different sections of the foundation rock may be subjected safely. Grouting pressures, the consistency of the grout, and the speed of injection are all important elements of foundation grouting.

Pressure Control.—Grouting pressures must be controlled carefully to insure against possible displacement of the rock. They must be adapted to the depth of hole, distance from exposed rock faces, and the character of the rock with respect, particularly, to open joints, lift seams, crush zones, etc. Generally, in considering the subject, and in the preparation of specifications, a distinction is observed between low-pressure grouting and high-pressure grouting. The former, with pressures ranging, in good rock, from about 50 lb per sq in. to perhaps 150 lb per sq in., is applicable to shallow hole grouting in general—say less than 40 ft in depth, and for the consolidation of upper strata rock to prevent the escape of grout from the deeper, high-pressure grouting to be done later. In the latter, pressures range generally from 200 lb per sq in. to 600 lb per sq in. There is danger of rock dislodgment with these extremely high pressures, and the utmost caution must be exercised in their application. Frequent observations, by means of precise levels, the tiltmeter, or other device, to detect displacement, should be made during the process of grouting wherever there is danger of rock movements. Although the proper depth of hole depends mainly upon the character of the rock structure, the deep holes usually have a length about one fourth the hydrostatic head at maximum water level behind the dam. Deeper holes are sometimes necessary.

Washing Out Seams.—Before grouting is begun, the grout hole should be cleaned with compressed air and water and at the same time effort should be made to clean the seams in the rock whenever that is practicable. For narrow seams filled with gouge or stiff clay it is neither practicable nor necessary; but for the wider seams, filled with silt or with products of solution, it often is practicable and should be done. The process involves the application of compressed air and water, as already mentioned, a carefully devised pattern of hole arrangement, and a hookup that permits a rapid reversal of flow between holes and through the seams. A good example of the successful application of this procedure is in the foundation treatment of Norris Dam.[11] The limiting pressures to be used in washing out grout holes and seams are practically the same as those previously defined for the grouting.

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[106] "Improving Foundation Rock for Dams," by James B. Hays, *Civil Engineering*, May, 1939, p. 309.

Grout Curtain.—Usually, although not always, a grout curtain parallel to, and near the upstream face, of the dam, effected by a single or a multiple line of grout holes, will suffice for such foundation consolidation as may be required for seepage reduction purposes. The depth of holes has already been discussed. The spacing of holes may vary from 2½ ft to 5 ft or more, depending upon the character of the rock and the depth of reservoir water behind the dam. Where more than a single line of grout holes is used, the holes should be staggered with those of the adjacent line. For high dams two or three grout lines, with shallow, intermediate, and deep holes, are the usual practice. The shallow holes are grouted first, then the intermediate, and finally the deep holes, the latter generally being deferred until a considerable height of concrete has been placed. Good evidence as to the effectiveness of grouting operations may usually be obtained by first grouting holes with a wide spacing, say 20 to 40 ft, and then using the intermediate holes as test holes by comparing their acceptance of grout with those first grouted. For a single line of deep holes, stage grouting is generally desirable and is usually specified.

Stage Grouting.—When a single line of deep or moderately deep holes is used for developing a grout curtain, and in other places when such holes are to be drilled (particularly where the seamy character of the rock demands special care in grouting), a process of stage or successive drilling and grouting is often adopted. This process consists of the following successive operations: Drilling the hole to a depth of from 10 ft to 40 ft, depending upon the character of the surface rock; grouting at that depth under relatively low pressures; cleaning the grout hole and drilling it to farther depth; grouting to the new depth under somewhat higher pressure, and so on until the ultimate depth of hole is reached; and finally grouting it under high pressure. The purpose and the advantage of stage drilling and grouting are that they consolidate the successive rock zones from the surface downward, under increasing grout pressures, thus making the final grouting under high pressure effective by insuring against undue escape of grout through the upper rock zones.

Special Grouting.—Special grouting is often required on steep abutments; where major surface seams, crevices, or fractures in the rock are in evidence; where consolidation of blocky rock is required; and, in fact, at any questionable sections of the foundation, wherever they may occur. On the abutments, however, where, in the absence of grouting or other consolidation or cutoff provisions, the percolation path may be relatively short, a program of special grouting is usually desirable. This condition, at minimum, involves shallow grouting back of the immediate faces of the abutments, but, if the rock is badly seamed and jointed, it may also involve grouting operations outside of, and beyond, the boundaries of the abutment contact areas. In some instances, too, it may be desirable to drive a tunnel into the abutment from which to accomplish the grouting and drainage required. Undue seepage, through and around the abutments, should be prevented, and the special grouting mentioned herein, together with properly placed concrete cutoff walls, is an effective means to that end.

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[11] *Engineering News-Record*, November 21, 1935; also *Proceedings*, Am. Soc. C. E., March, 1940, p. 385.

Drainage.—There are few, if any, high masonry dams that do not require some provision for drainage, and such provision is now the practically universal practice in dam construction. Its purposes are to relieve upward pressure, thereby increasing the stability of the dam, and to pick up and safely conduct to the river channel below the downstream toe of the dam any seepage water that may find its way through or beneath the dam. It is accomplished, usually, by means of a series of drainage holes, 3 in. to 6 in. in diameter, about 10 ft on centers, and drilled into the foundation rock to depths of from 25 ft to 50 ft or more, depending upon the character of the rock structure and the depth of the adjacent grout holes. The holes are located on a line parallel to, and a short distance downstream from, the grout curtain. One or more additional lines of drainage holes may be required in some instances. The seepage water that wells up through the drainage holes, usually into a drainage gallery in the dam, is delivered, through transverse galleries and lesser conduits, to the downstream face of the dam close to the surface of tailwater. In placing the base concrete of the dam, metal pipes may be installed, or holes may be formed otherwise, to facilitate the subsequent drilling of the drainage holes, and these holes should not be drilled until all grout holes, which might possibly be connected by seams to the drainage holes, have been drilled and grouted.

Special Abutment Treatment

In general, the problem of preparing abutments is based on the same requirements as that of preparing the remainder of the foundation. As a matter of fact, the dividing line between the foundation and the abutment is frequently an imaginary or arbitrary one, and depends largely on how the abutment slope flattens out to meet the generally horizontal position of the bottom area. In a broad sense, the entire area upon which the dam rests, and the adjacent areas which are directly affected by the proposed structure, should be considered as foundation. In this section, however, the term "abutment" refers only to the steeper foundation slopes on either side of the bottom area.

Horizontal, or nearly horizontal, seams or solution channels, if any, are usually exposed by the abutment excavation and therefore are accessible for methods of direct treatment. Where such seams are of sufficient importance, they may be tunneled out and filled with concrete, or, where control of seepage is the main consideration, several cutoff drifts properly spaced upstream and downstream may be effective. In such cases a drift at the upstream face of the dam should be driven as far as may be necessary to force a percolation path of sufficient length to prevent undue seepage, or to a point where extension by grouting might be more practicable. In some cases, depending upon the character of the seam, this one cutoff may be sufficient; in other cases it may be desirable to supplement it by others so spaced as to extend further the path of any seepage that might find its way past the upstream cutoff. Sealing off the faces of these seams by substantial concrete plugs is usually desirable, and often this makes it possible to do effective grouting behind the seal. In seamy rocks this control of seepage, so as to force it out and around the main structure of the dam and well into the abutment, is one of the important factors in abutment treatment. Frequently the work required to accomplish this must extend well out beyond the limits of the structure.

Abutment excavation necessarily involves steep slopes or stepped benches. Steep slopes, although unavoidable, are undesirable because they cause unequal shrinkage within short distances in the concrete of the dam. Stepped benches are undesirable for the same reason, and require also that their edges be chamfered off to avoid a tendency to originate cracks. These are unfavorable conditions that occur to some extent in every dam abutment, and must be considered both in the preparation of the abutment foundation and in the design and construction of the dam itself. If the undesirability of steep slopes and benches is kept clearly in mind, together with the reasons why they are objectionable, then the measures adopted to meet the situation are most likely to be effective. These measures usually are limited to a combination of adjusting slopes, chamfering edges of benches, locating contraction joints so as to best fit the situation, and so placing the concrete that the effect of unequal shrinkage, or the tendency to crack from other causes, will be most effectively counteracted. Fig. 6 shows a part of one of the abutments of Grand Coulee Dam during the process of scaling and trimming.

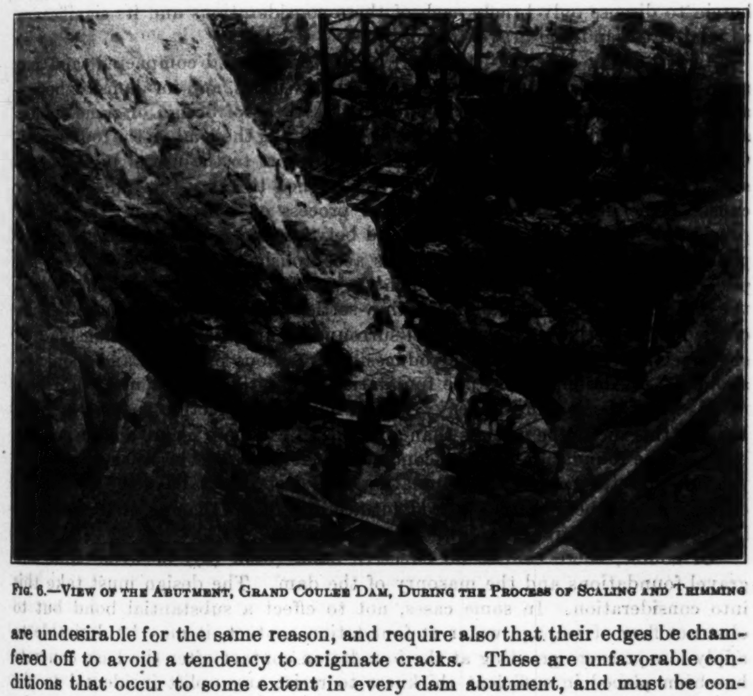

FIG. 6.—VIEW OF THE ABUTMENT, GRAND COULEE DAM, DURING THE PROCESS OF SCALING AND TRIMMING

SPECIAL CONDITIONS

Sand and Gravel Foundations for Low Dams.—Consideration of building low masonry dams on sand and gravel foundations opens up a special field in dam design and construction. As far as preparation of foundation is concerned, the main considerations (see itemized considerations at the beginning of the paper) are the same as those for higher dams on rock foundations, but the very nature of these sand and gravel foundations calls for special study of these considerations as they apply to foundations of this type. It is the purpose herein to discuss only briefly each of these considerations and its significance as related to sand and gravel foundations.

As to bearing capacity (item (1)), a well-graded and compacted sand and gravel foundation may be sufficient for very low dams, and for dams of moderate height which follow the slab and buttress type of design or some similar special design which reduces unit stress by spreading the load on the foundation. Otherwise, in many cases, it is necessary to resort to the use of piles, and in such cases this pile foundation must be designed to carry the full load. Because of the necessary sequence in the process of construction, there is no practicable way of distributing the load between the sand and gravel and the piles, except to the extent that this is accomplished by friction between the piles and the material through which they are driven. The piles must take the entire superimposed load unless they settle sufficiently to transfer a portion of the load directly to the surface of surrounding material, but obviously, even slight settlement of the dam introduces other serious problems and should be avoided. In designing the pile foundation, a limitation that must be recognized, when the loading is heavy, is the minimum effective spacing of piles. Usually this should be not less than about 3 ft. To help secure a close contact between the foundation and the concrete of the dam, and in order to effect all possible presettlement of the foundation, it is desirable to have the foundation dewatered well below the prepared surface during the placing of the first few layers of concrete around and above the piles.

Of course, there can be no effective bond (item (2)) between sand and gravel foundations and the masonry of the dam. The design must take this into consideration. In some cases, not to effect a substantial bond but to obstruct flow of seepage water at foundation contact, it may be desirable to apply low-pressure grouting at the foundation contact after the base concrete has been placed in sufficient thickness to resist any uplift incidental to the grouting. To prevent the escape of the grout around the edges, resort may be had to "dry packing" or "stock ramming."

Resistance to sliding (item (3)) must be considered carefully. Where piling is not used, sufficient resistance may be developed by broadening the base, by setting the dam well into the foundation, or by substantial cutoffs properly designed, and properly located, so as to be effective in this respect.

Seepage (item (5)) is usually controlled by sheet-pile cutoffs along the upstream face of the dam. The piling preferably should be driven to rock or to a layer of impervious material. If this is not practicable, then the sheet-pile cutoff should be of sufficient depth to insure that the shortest possible seepage path will be long enough to prevent movement of the finer materials in the foundation. Substantial, impervious, upstream blankets are effective aids in the control of seepage, in that they lengthen the path of seepage travel. These upstream blankets, usually of clayey earth, compacted by rolling or other effective means, are relatively inexpensive and when properly designed and constructed are very reliable. Concrete aprons are sometimes used for the same purpose. Special attention must be given, in the design, to the junction between these blankets or aprons and the masonry of the dam, so that a tight contact can be made there which will not be destroyed by slight settlement, and will not permit incipient leaks or piping.

In foundations of this type, a common measure of uplift pressure (item (4)) at any point is that obtained from an hydraulic grade line starting at the elevation of full reservoir at the upstream edge of the upstream blanket, or at the upstream face of the dam if there is no blanket, ending at the elevation of tailwater at the downstream limit of the structure, and adjusted between those points for such effects as may be conservatively assumed to result from sheet-pile or other cutoffs. In special cases—say, beneath the downstream third of the structure—downstream drainage may be desirable in order to insure that there will be no backing up of pressures, within the drained area, above the limits indicated by the theoretical hydraulic grade line.

Where a river-channel spillway section is a part of the design of a dam on sand and gravel foundations, as is often the case, protection against scour (item (6)) downstream from the spillway is of utmost importance. It is best controlled by a well-designed stilling pool, so developed as to be effective within the complete range of headwater and tailwater elevations. The downstream lips of these stilling pools are sometimes protected from undercutting by rows of round piling closely spaced, or by rock or concrete aprons; but a more effective method of protection is by a lip so designed as to form a ground roller that tends to pull back the loose channel material toward the edge of the lip—the exact opposite of undercutting. Lip designs to accomplish this result are quite dependable; in working them out, model tests are of great value.

Cavernous Rocks.—Cavernous rocks present many special problems which must be considered in connection with their geological structure; loading conditions to be met; extent and character of solution channels and their connections; and permissible loss of water from the reservoir, from the viewpoints of its value, of actual danger, or of the psychological effect of the seepage around or below the dam. In such rocks seepage may or may not endanger the structure, but even if it is not dangerous, such seepage, uncontrolled, may be extremely undesirable. By careful (and usually quite expensive) work, after adequate exploration and study, even cavernous foundations may often be treated so as to be acceptable for dams of moderate or intermediate heights; but it is obvious that such foundations are to be avoided if possible, and that they add a large, as well as an uncertain, element to the cost of the structure. Limestones, and basalts or other volcanic formations, offer the greatest possibility of cavernous structure.

Close drilling to sufficient depth usually is required to locate extensive caverns or solution channels. They can then be reached by large-diameter drills, or otherwise, cleaned out as may be necessary, and plugged with concrete or some effective substitute. Spacing of drill holes for a grout curtain in cavernous rock may need to be as close as 1 or 2 ft, in order not to miss a space that might defeat the object of the curtain.

Shales and Other Stratified Rocks.—Shales and other stratified rocks require special study because of their diverse characteristics, and because of the liability of finding soft or permeable seams between the harder layers. A careful subsurface exploration is necessary to detect such weaknesses. The harder shales and most other stratified rocks are usually sufficient as to bearing capacity except for high dams.

Bonding to shales is best accomplished by a series of cutoffs to break up the continuity of the stratification, and by leaving the final trimming until just before covering with concrete, as most shales are likely to disintegrate quickly upon exposure. Other sedimentary rocks may or may not require special treatment for securing a tight bond, depending upon their characteristics.

Protection against sliding must be considered carefully in dealing with such rocks. Often the best plan is to set the dam deep into its foundation, so that downstream shoulders will hold it against movement in that direction. In such cases it is very essential that these supporting shoulders be protected against scour or disintegration. This may be accomplished by covering with concrete or other paving to protect against scour, or by covering with earth, a weather-proof coating, or some other similarly effective method to guard against disintegration.

Seepage may be controlled by cutoffs and, in some instances, by grouting, as already explained for other classes of rock. Whether grouting will be effective depends upon the character of the seams. In these types of rock, control of grout pressures is of special importance. Usually, by close spacing of holes, a dependable grout curtain can be secured, even if low pressures must be used and although few of the holes may take any appreciable quantity of grout.

Because of the position of seams and bedding planes in sedimentary rocks, uplift pressure must be given special consideration. Measures adopted for control of seepage are helpful in this respect, but horizontal seams, which may be continuous under the entire foundation, present an element of weakness that should not be ignored. Drainage holes, properly placed in such foundations, often give assurance of adequate control within limits anticipated in the design.

Stratified rocks are likely to be weak against attack by scour. Stilling pools and aprons should be designed with this in mind. Shales should be protected from exposure far enough beyond structure limits to insure against the effect of progressive disintegration back toward the dam.

CONCLUSION

It is the hope of the writers that this paper will succeed in centering the readers' attention on the basic considerations involved in the examination and preparation of foundations for masonry dams, as outlined herein. It is hoped also that sufficient emphasis has been given to the more usual conditions and methods of treatment to indicate the importance of a careful independent study of each foundation problem, to the end that no significant condition, and no necessary precaution, will be overlooked.

GEOLOGICAL PROBLEMS OF DAMS

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SYNOPSIS

The geological and foundation problems involved in the economical construction of safe dams, and the possibilities of geological investigation, are discussed in this paper. The different stages of a complete geological investigation and the objectives of each stage are also discussed. Every branch of geology may be involved in investigation of a dam site and in addition many allied techniques and methods may be necessary. The paper is confined to the consideration of geological and foundation problems of masonry dams founded upon rock.

Foundation problems vary for different types of foundation rock and for different types of geologic structure. These problems are illustrated by examples of successful dams and dam failures on different types of rock and geologic structure. The purpose of these descriptions of dam failures is to point out the various geologic conditions which may cause failure and not to renew discussion of the conditions pertaining to each individual failure. The thesis of this paper is that the essential foundation conditions can be determined in advance, that at most sites safe dams can be built, and that dam failures are not unavoidable.

INTRODUCTION

Every dam rests upon geological formations and geological structures which, before the dam is built, are stable, or relatively so, but which, when the dam is finished and the reservoir is filled, are subjected to new and different stresses and conditions. The engineer must so design his dam in relation to these formations that they will continue to be stable under all possible conditions.

The problem derives much of its complexity from the fact that geological formations and structures are very seldom uniform for any considerable extent. They change vertically and horizontally, sometimes with great rapidity, sometimes gradually. The true conditions at a dam site must be determined and the design of the dam must conform to the conditions imposed by Nature. If these conditions are not met, the dam may fail. It may slide on a slippery bedding seam just below the foundations, as happened at Austin, Pa.; or the raised ground-water level may cause softening of the rock, as occurred at the St. Francis Dam, in California; or the water may pass under the dam through fissures and caverns, as at Hales Bar Dam on the Tennessee River, in Tennessee, or any one of a thousand possibilities may occur to destroy the dam or reduce its value. Not only is it necessary to determine the existing conditions of the rocks, but the possible effects of new stresses, new ground-water conditions, and new exposures to weathering and oxidation must be foreseen. Formations which are now stable may become unstable under new ground-water conditions or when exposed to weathering.

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For the satisfactory interpretation of foundation conditions, all the resources of geological knowledge are necessary. Since it is seldom feasible to obtain all the desirable information about subsurface conditions, it is necessary to use geological principles and broad experience based upon work with every conceivable type of geological formation and geological structure in order to estimate conditions as accurately as possible. In reaching conclusions about subsurface conditions, the engineering geologist must make certain that no possible variation from his estimate can be such as to endanger the project. He must weigh the probability of accuracy of his statements, point out in what way actual facts may vary from them and what the effect of such variation would be, and must show what the worst possible conditions are. The practice of engineering geology, therefore, requires an unusually broad scope of geological training and knowledge of the constantly developing related sciences and techniques. Above all, it requires the ability to make important decisions on insufficient data with the assurance that those decisions are the best possible. Furthermore, in no type of geological work are the geologist's conclusions checked as remorselessly as when the excavations for a dam expose to view the underground conditions he has forecast, and when the filling of the reservoir puts a final test to his conclusions.

The use of geology in dam site investigation is now almost universal, but there is often failure to recognize the extreme importance of experience in this type of work. It is too often assumed that any geologist can give satisfactory results or that the geologist can be trained on the job and that prior experience is not necessary. A geologist trained on the job may make valuable observations, but, when any unusual condition arises, there is grave danger of serious mistakes.

To make a correct interpretation of subsurface conditions with whatever data are ascertainable, the engineering geologist must use methods of reasoning and of work different from those to which the engineer is most accustomed, for the precise and mathematical methods of engineering reasoning can seldom be used by the geologist. It is true that recent developments in geological and related sciences have given the geologist tools of quantitative measurement which partake of the methods with which the engineer is familiar; but the geologist must use these tools with great care since, if used without sufficient experience and knowledge, they may be dangerous. Both geologist and engineer, when questions relating to subsurface conditions are concerned, must be on their guard against a facile acceptance of seemingly precise quantitative results, for conclusions based on them can be no more accurate than the interpretation of subsurface conditions to which they are applied.

The duty of the geologist in investigations of dam sites should be to discover and make clear to the engineers in charge, as accurately and completely as possible, all the geological conditions and factors which may have any relation to the dam and to the construction work. When it is impracticable to determine all the details of the geological conditions he must point out the worst possible conditions for which the engineer must design to insure safety of the dam. It should not be the duty of the geologist to make engineering decisions based upon geological facts. When the geologist has presented to the engineer a full description of geological conditions, the engineer can meet those conditions or decide whether it is not economical to meet them.

The geological investigation of dam sites naturally divides itself into three parts with different objectives: First, a reconnaissance investigation during the preliminary stage; second, the detailed investigation in the design stage; and third, continuing geological observation during the construction stage.

The reconnaissance investigation should be confined to a brief surface study without borings and should be concentrated upon the physiography, visible bedrock geology, and the overburden. The important objectives are: To determine the presence or absence of buried valleys or gorges; to make an approximate estimate of the probable depth to rock; to recognize the rocks, with especial attention to those which are weak or soluble; and to understand in a general way the geological structure, with special attention to faults, zones of weakness, and possible sliding planes. This stage makes the greatest demand upon experience, and a very broad knowledge of geological conditions and engineering requirements is necessary. Such a reconnaissance investigation cannot give the detailed information necessary for design, but it should make possible the choice of the site with the most favorable foundation conditions and should make known the general type, and difficulty, of the foundation problems. Such an investigation can make reasonably certain that the project is economically feasible. In this stage great savings in time and money are possible by discarding unfavorable sites before expensive and time-consuming surveys and studies have been made. Greater use of experienced engineering geologists at this stage is desirable.

The second stage requires a very thorough geological examination of the dam site and vicinity and a thorough subsurface investigation of the site to ascertain whether formations are continuous and homogeneous, or are lenticular, or vary in character, and to work out the details of geological structure and their relation to the dam in order to present to the engineers all pertinent facts about the formations and their structure and to answer all questions of the engineers. This stage of the investigation should give the information needed in the final design of the dam. The engineering geologist is used most in this stage.

After the work has passed beyond the second stage, and the geologist has satisfied himself and the engineers that the natural foundation conditions are such that a safe and satisfactory dam can be built at the site, it is often assumed by engineers that, unless obvious trouble arises, a geologist is no longer needed. However, frequent geological observations during the opening up of foundation excavations will insure that geological details are properly provided for, and may make possible a saving of excavation and concrete. The forecasts made by the geologist during the earlier stages cannot, from the nature of the problem, be 100% correct, and it is very important, therefore, to recognize promptly any variations from forecasted conditions and decide whether they affect the plans for the dam. Therefore, the continued cooperation of geologist and engineer until the foundations are completely excavated and examined, and all the problems met, will prove most efficient and satisfactory.

METHODS OF GEOLOGICAL INVESTIGATION OF DAM SITES

In the preliminary investigation to select the best site and to determine the feasibility of the project, much information can be obtained on many problems, such as the location of buried gorges or concealed faults, by a physiographic study of the area. Unless the geological and physiographic history of the valley is understood, there is danger that some important facts will escape discovery by even the most elaborate drilling program. In poorly mapped areas, aerial observation or the study of aerial photographs with the stereoscope may be very helpful. Where subsurface information is necessary, much drilling may be saved by using geophysical methods. The seismic method is especially suitable for detecting buried gorges since it gives a profile of the bedrock surface. Finally, when the exact site has been chosen and it is desired to develop the details of the site, test drilling will be necessary; but the geological and geophysical investigations will have made it possible to locate the borings strategically so that the necessary information can be obtained by a minimum number of borings. Such a program utilizes the rapid and less expensive procedures of geology and geophysics to keep to a minimum the more expensive and slower method of drilling.

In the second stage of the investigation—that is, the thorough study of the dam site—the geologist's first step is geological mapping of the site and vicinity, the area depending upon the complexity of the geology and other factors. This involves a very careful study to detect all features that may affect the dam, such as zones of depth, weathering, and soluble rocks, and their relation to the dam. If soluble rocks are present, every means must be used to locate underground solution channels and caves. A thorough study of ground-water levels and movements will be useful in estimating seepage around the ends of the dam. Location of test borings with regard to the geological structures may make possible a saving in number and footage of borings and a consequent saving in cost.

The geologist must use every available tool and allied science in solving the complicated problems. In addition to the strictly geological methods, he will need to obtain subsurface information by core borings and test pits and may use shafts and tunnels. Bearing tests and other field tests upon the rock in place may be desirable. Wherever there is question of leakage through the rocks under or around the dam, a study of the permeability of the rocks by means of water tests on drill holes becomes important. In addition to these many field methods, some of the following laboratory tests and procedures may be necessary, depending upon the rocks and structures: Determination of true and apparent specific gravity, porosity, and moisture content; solubility and slaking tests; petrographic examination; chemical analyses; consolidation tests on specimens of shale; compression and shearing tests; and determination of modulus of elasticity. Not all of the field and laboratory tests are needed at any one dam site, and the use of all of them promiscuously would be a serious waste of time and money. New methods of value in investigations of dam sites have been developed in recent years; but further development and better understanding of certain problems are desirable and will aid in making more practicable thorough investigations of dam sites.

Core borings have long been one of the most useful tools for investigating the rock at dam sites, and it is sometimes helpful, in studying complex geological structures, to record the boring results on three-dimensional models. A recent development of great value is the use of large drill holes from 2 to 3 ft in diameter into which the geologist can descend and study the section of the foundation in its natural conditions and detect faults and zones of weakness. One of the first uses of these holes was at the Prettyboy Dam, in Maryland, in 1931, but they had previously been used in quarries.[13] Various devices have been tried for locating open cracks in smaller drill holes. A periscope was used at Norris Dam, in Tennessee, and at Conchas Dam, in New Mexico, and a feeler which indicates any recesses in the walls of the holes was used also at Norris Dam.[14]

Geophysical prospecting may be used in preliminary investigation and in some cases in the more detailed study of dam sites. The electrical method was used first, but the seismic method has been used at many sites.[14a] Both the seismic and electrical methods have been used successfully under water.[15] Submarine acoustic methods have been used to detect leaks from reservoirs. Geophysical methods were first applied to study of dam sites at the site of the Fifteen Mile Falls Dam, in New Hampshire, to determine the depth to bedrock.[16] The electrical method has been used successfully to study geologic structures and locate faults and decomposed zones.[17] There are several techniques, some of which are described in the articles referred to and in others.[18]

A modification of electrical prospecting known as "electrical coring" has been used extensively in the oil fields and has more recently been applied to dam site investigation.[19] In core drilling it is often impossible to obtain core at the most critical places, but "electrical coring" gives information about the rock at those places where core cannot be obtained. If the method is correlated with a few actual cores from core drill holes, it is possible to identify the same strata in holes drilled without coring. Information is also obtained about the porosity of the rocks, liquid pressure in the rocks, and strike and dip of the rocks. If there is a fractured zone from which it was impossible to obtain core, the nature of this zone can be determined.

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[13] "Weathering and Albitization of the Wissahickon Schist at the Prettyboy Dam, Baltimore County, Maryland," by Joseph T. Singewald, Jr., *Bulletin*, Geological Soc. of America, Vol. 43, 1932, p. 450.

[14] "Thousands of Holes Grouted Under Norris Dam," *Engineering News-Record*, November 21, 1935, p. 700; also "Unique Devices Developed to Aid Dam Foundation Grouting," *Engineering News-Record*, August 8, 1935, p. 191.

[14a] "Seismic Refraction Methods as Applied to Shallow Overburdens," by F. L. Partlo and Jerry H. Service, *Transactions*, A. I. M. E., Vol. 110, 1934, pp. 473–492; "Practical Seismology and Seismic Prospecting," by L. D. Leet, D. Appleton-Century Co., Inc., pp. 347–399.

[15] "Electrical Exploration of Water Covered Areas," by C. and M. Schlumberger and E. G. Leonardon, *Contribution No. 71*, A. I. M. E., 1934; "Geophysical Prospecting, Mining and Metallurgy," by Sherwin F. Kelly, January, 1938, pp. 15–17; and "Étude Géophysique Sous-Marine Exécutée dans le Port d'Alger," by Conrad Schlumberger and Pierre J.-M. Renaud, *Annales des Ponts et Chaussées*, IV, 1933.

[16] "Electrical Prospecting Applied to Foundation Problems," by Irving B. Crosby and E. G. Leonardon, *Technical Paper No. 131*, A. I. M. E., 1928; also "Electrical Subsoil Exploration and the Civil Engineer," by Irving B. Crosby and Sherwin F. Kelly, *Engineering News-Record*, February 14, 1929, pp. 270–273.

[17] "Some Application of Potential Methods to Structural Studies," by E. G. Leonardon and Sherwin F. Kelly, *Technical Publication No. 115*, A. I. M. E.; also "Application des Méthodes de Prospection Électrique à l'étude des fondations de Hauts Barrages et des Ouvrages Annexes," by Maurice Lugeon and Conrad Schlumberger, *Le Génie Civil*, August 6, 1932, pp. 134–137.

[18] "Application of Electrical Prospecting to the Study of Dam Sites," by C. and M. Schlumberger, 2d Cong. on Large Dams, Vol. IV, pp. 67–80, Washington, D. C., 1936; "Topographical Study of a Hidden Bedrock Surface by Resistivity Measurements," by E. G. Leonardon, *Engineering Journal* (Canada), June, 1931; and "Electrical Exploration Applied to Geological Problems in Civil Engineering," by E. G. Leonardon, *Technical Publications No. 407*, A. I. M. E., 1931.

[19] "Electrical Coring; a Method of Determining Bottom-Hole Data by Electrical Measurements," by C. and M. Schlumberger and E. G. Leonardon, *Technical Publications No. 462*, A. I. M. E., New York, 1932; also "A New Contribution to Sub-Surface Studies by Means of Electrical Measurements in Drill Holes," by the same authors, *Technical Paper No. 503*, A. I. M. E., New York, 1933.

In addition to the field examination and the field tests, many laboratory tests upon specimens of the foundation materials may be necessary as part of the geological investigation.

Direct shearing tests made on rocks in any of the direct shearing devices do not give the true shearing resistance since progressive failure takes place due to concentrated stress. The approximate shearing resistance can be computed from unconfined compression tests on cylinders with length at least twice the diameter. The true shearing resistance can be computed from properly conducted triaxial compression tests on cylinders, with length at least twice the diameter, using different confining pressures.[20] Attempts to estimate from triaxial compression tests the actual shearing resistance of rock in the ground might give too favorable results because it is impossible to reproduce, in the laboratory, the effect of cracks in the bedrock. The results of shearing and compression tests should be interpreted by the geologist after observation of the rocks under load in Nature and after study of conditions as they will exist in the foundations.

GEOLOGY OF DAM SITES

The problems of masonry dams on rock vary with the type of rock, and the problems of each broad rock type will be discussed separately and illustrated by examples of successful dams and of failures. However, there are problems dependent upon structure or other geological characteristics, independent of the rock type, and these will be discussed first. The first problem is the selection of the dam site with the best bedrock profile, and the most difficult part of this problem is to detect any buried gorge in the bedrock surface, either at or near the dam site, which might be missed in the preliminary test drilling. Such a gorge was found at one site, between drill holes spaced 50 ft apart.

Buried valleys or gorges are most common in glaciated regions where the drainage was disarranged by the ice sheet. The postglacial streams have often cut down upon rock ridges, ignoring their old valleys now filled with glacial drift. This is generally the explanation in the glaciated regions for a stream with rapids or falls in a narrow rock gorge with wider valleys above and below. Each type of buried gorge or valley is associated with a definite geological history and is indicated by physiographic features which serve as valuable clues to the geologist experienced in their interpretation. Sometimes these features will show up well from the air or in aerial photographs.

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[20] "The Shearing Resistance of Soils," by L. Jurgenson, *Journal*, Boston Soc. of Civ. Engrs., July, 1934, pp. 242–275; "Versuche ueber Festigkeitseigenschaften von Sand in dreiachsigen Spannungszustand Wasserwirtschaft und Technik," by W. Bernatzik, 1935, p. 184; "Report on an Apparatus for Consummate Investigation of Mechanical Properties of Soils," by W. Kjellman, *Proceedings*, International Conference on Soil Mechanics and Foundation Eng., June, 1936, Vol. 3, pp. 16–20; and "Ein Grundgesetz der Tommechanik und sein experimenteller Beweis," by L. Rendulic, *Der Bauingenieur*, 1937, Nos. 31–32.

Buried valleys may underlie the reservoir site and lead from it at points far from the dam site. They will not directly affect the dam, but they may cause serious leakage from the reservoir. The detection of possible leakage from the reservoir, except at the dam site, whether through buried valleys, through faulted or crushed zones, through solution channels in soluble rocks, through lava tunnels, or through porous rocks, presents important and difficult problems which cannot be discussed at length in this paper. They are problems, however, that are especially suited to geological and physiographic study, and in which geophysical methods and ground-water studies may be very valuable. Leakage at the dam site or elsewhere in the reservoir basin, although it may not endanger the dam, may render a good dam useless by preventing storage of water in the reservoir. The seriousness of any given amount of leakage is governed by the use for which the reservoir is intended. Decision as to whether leakage will continue to remain harmless or become serious is a geological problem requiring careful study and great experience.

Among the geological structures which may be of importance in relation to dams are: Faults, crush zones, joints and zones of close jointing, bedding seams, sharp folds, zones of deep weathering, zones of alteration, solution channels and cavities, and lava tunnels. A fault is a fracture along which there has been differential movement, and its relation to dams may be important or relatively unimportant depending upon its condition. Recent faults raise the question of future movement with accompanying earthquake action. A fault may be an open fissure, or it may be filled with gouge or crushed rock. In either case it is important in regard to leakage as well as in regard to structural strength. The presence of gouge increases the difficulty of grouting the fracture. Although a number of dams have been built across faults, there is no known record of a masonry dam across a live fault during an earthquake. The Crystal Springs concrete gravity dam, of the San Francisco water supply, is less than a quarter of a mile from, and parallel to, the San Andreas fault on which the earthquake of 1906 occurred. The dam was not injured by the earthquake, but it was not intersected by the fault. The Rodriguez, Ambursen-type dam on the Tiajuana River, in Mexico, was built across a dead fault which intersected the axis of the dam under the stream bed.[21]

Morris Dam in San Gabriel Canyon, California, is a gravity, concrete dam across a fault. Geologic evidence in the form of undisturbed river gravels indicated that there had been no movement on the fault for thousands of years but provision was made for possible future movements by the construction of an earthquake joint. Examination of striæ on the walls of the fault indicated that movement had always been in the same direction. An open joint has been provided along the trace of the fault between blocks 8 and 9, as described in 1934 by Samuel B. Morris, M. Am. Soc. C. E., and Cecil E. Pearce, Assoc. M. Am. Soc. C. E.[22] The joint will provide for differential movement of 6.55 ft along the fault.

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[21] "Unique Cutoff Construction and Arched Foundation Features of Rodriguez Dam," *Engineering News-Record*, October 16, 1930, pp. 600–604; also "Foundation Treatment at Rodriguez Dam," by Charles P. Williams, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 99 (1934), pp. 295–305.

[22] "A Concrete Gravity Dam for a Faulted Mountainous Area," by Samuel B. Morris and Cecil E. Pearce, *Engineering News-Record*, December 27, 1934, pp. 823–827.

Joints are fractures along which there has not been appreciable movement. They are generally nearly tight at depth but are usually somewhat open near the surface. They intersect practically all rocks and are often in systems, with the joints of each system approximately parallel to each other. Joints cut up the rock into blocks of various sizes, decrease the structural strength of the rock, and are important in regard to seepage. The relation of the direction of the principal joints to the axis of the dam is important in connection with foundation stability, seepage, and grouting. Consequently, it is important for the geologist to determine the relation of the joints and other fractures to the dam and their effect upon it in regard to stability and seepage.

The different beds of stratified rock are separated by bedding planes. The rock may be continuous across these planes, but they are zones of weakness. In some cases they are shale seams; occasionally they are open cracks, in which case they are important in regard to seepage. Folding of the strata may have tended to open bedding seams and joints, especially along the crests of anticlines, and may have formed transverse joints. Joint cracks and bedding seams are especially important in regard to leakage under or around the dam.

The resistance of rocks to erosion is important in regard to the effect of overflow on the rocks below the spillway. Their resistance depends partly on their strength and hardness but even more on their structure. Closely jointed and thin-bedded rocks are especially susceptible to erosion since the rushing water enters joint cracks and bedding seams, exerts hydrostatic pressure, and pries off slabs and loosens blocks. The actual abrasion of flowing water, even when armed with sand and gravel, is relatively unimportant compared to its quarrying action where the rock is fractured.

The increasing height of dams emphasizes the importance of the elastic properties of rocks and of the strain conditions existing in the rocks. In some cases tests upon specimens of rock, for the modulus of elasticity, and upon the rock in place (as in tunnels), for residual strain, should receive attention in order that the design of the dam may meet the conditions imposed by the different elastic properties and strain conditions of the rocks and of the concrete. Expansion of rocks upon relief of stress is a well-known occurrence in quarries, and residual strain may be expected in rocks in deep canyons. The greatest contrast in the elastic properties of rocks and concrete probably occurs with some uncemented shales, and in such cases the rock will deform under load differently from the concrete. Excessive deformation of the foundation rock may injure grout cutoffs and the bond between rock and concrete. The practical importance of the elastic properties of rocks is greatest in the foundations of high gravity dams and the abutments of arch dams.

Arch dams make especially severe demands upon the abutments since the rock in these must be strong enough to withstand the thrust of the arch. The structure of the rock is unusually important here, and rock which in itself is amply strong may be intersected by joints or faults in such a way that the mass as a whole cannot offer the necessary resistance to the thrust. Therefore, the exact condition of the rock and of its structure must be determined and the effects of the new stresses upon it must be studied. The effects of deep weathering and alteration must receive careful study also. These effects tend to be more serious in abutments outside the glaciated regions. The possibility of earthquakes, and the presence of any faults upon which movement might be renewed, become of unusual importance with this type of dam.

DAMS ON SHALE

The term shale is used in the broad sense in this paper, and it is not confined to those that have a well-developed lamination or thin-bedded stratification, but includes those unlaminated, argillaceous rocks with no apparent stratification which may appropriately be called siltstones or mudstones. Shales are formed by the compaction of clay and silt with, or without, cementation. Depending upon the degree of compaction and cementation, they may vary from a very soft, weak rock, little different from clay, to a strong, weather-resisting rock. They may be divided into shales that have not been cemented, but have been compacted only, and shales that have been cemented in addition to compaction. The practical problems of dams on shales depend upon the type of shale and, for the laminated shales, they are: Sliding, uplift, strength of the shale, seepage through fractures, and the preparation of foundations. With uncemented shales the important problems are: Strength of the shale, sliding, settlement due to additional compaction, elastic deformation, prevention of disintegration, and preparation of foundations. The uncemented shales weather very rapidly due to the fact that they dry out and crack when exposed to the air and disintegrate when wet again. The cemented shales weather more slowly and do not disintegrate with alternate drying and wetting, but there are often poorly cemented strata which weather rapidly.

Shale probably requires greater care and more precautions in the preparation of the foundation than any other rock. If the shale is laminated, and especially if the stratification is flat, care must be used to remove all weathered and loose, or partly loose, slabs without causing an unnecessary amount of excavation. Blasting may often injure the shale and require much deeper excavation than would otherwise be necessary. If the shale is uncemented, great care must be exercised to prevent drying and disintegration when exposed. If the surface of an uncemented shale is allowed to dry for even a short time before concrete is placed upon it, the water of the concrete causes the surface of the shale to slake and form a permanent layer of mud between the concrete and the soft clay. This condition can be avoided by reducing the time of exposure, and by coating the freshly exposed surface of the shale with an asphaltic or other protective coating. Such protective coatings have been used with satisfaction at the Muskingum Dams, in Ohio, Fort Peck Dam, in Montana, Conchas Dam, and elsewhere. Experience and experiments have proved that the bond between the concrete and uncemented shales is improved by coating the shale with a waterproof preparation.

Shales are relatively watertight since joints are often not well developed in them and the bedding planes are generally tight. In the case of uncemented shales, existing cracks tend to be closed by plastic flow due to the weight of the dam. Open fractures are more likely to occur in the cemented shales and, since some of the beds are often less well cemented and are subject to disintegration and erosion, the closing of all cracks by grouting is important. In laminated shales, and in fact in all stratified formations where there is a possibility of open bedding seams, it is important to avoid excessive grout pressures since pressure, whether from grout or test water, in a partly open bedding seam tends to lift the overlying rock and open the seam farther. Cases of actual measured uplift of structures have been observed. Obviously, if the pressure is restricted to the net weight of the overlying material, there can be no uplift and this is a safe rule. A geological investigation of the condition of flat seams should give information on this point. Grout pressure sufficient to cause uplift may actually injure the foundations as well as waste grout.

The compressive strength of shale varies greatly, depending upon the amount of compaction and cementation. Unconfined compression tests on cylinders of an uncemented shale from Texas showed compressive strengths ranging from 156 to 785 lb per sq in., and similar tests on partly cemented shales from the "coal measures" of West Virginia gave results varying from 3,036 to 4,925 lb per sq in. These values show the great variation in crushing strength of different shales and the necessity of testing the different beds of each specific shale. The modulus of elasticity for an uncemented shale in Texas varied from 49,800 to 207,000 lb per sq in. at 100 lb per sq in. load, which is far below the value for concrete.

Resistance to sliding is of utmost importance with dams on laminated shale. The irregularities of the contact between concrete and shale can be made sufficient to prevent sliding of the concrete upon the shale. The problem, therefore, becomes one of the sliding of one bed of shale upon another, which depends upon the shearing resistance of the shale along the bedding planes, or the coefficient of friction if the bedding plane is a complete fracture. Geologic examination should indicate the extent to which bedding planes are actual fractures. Then it can be seen whether the problem is one of shearing or of friction of shale upon shale. The inclination of the bedding has a most important effect upon the resistance to sliding since, if the bedding is flat or is inclined slightly downstream, resistance to sliding will be at a minimum; but if the bedding is inclined upstream, resistance to sliding will be increased. If the dip is even moderately steep, sliding will involve shearing the shale across the beds, which will offer much greater resistance than shearing along the bedding planes. The deeper the base of the dam is set into the shale, the greater will be the resistance to sliding, since this will throw the potential shear planes deeper into the shale and necessitate shearing across the bedding. This may be accomplished without deepening the main part of the foundation excavation by constructing a reinforced anchoring wall as was done at Lake Lynn Dam on the Cheat River, in West Virginia, or by sinking reinforced concrete dowel pins into the shale as was done at the spillway gate structure at Fort Peck Dam.

Uncemented shales have undergone deformation due to the irregular removal of load as in the erosion of a canyon. This may have resulted in the formation of numerous small shear fractures which weaken the shale, and this possibility must be considered in the geological investigation. Uncemented shale, like clay, is subject to additional compaction resulting in squeezing out water when it is subjected to additional load. Therefore, if a dam is built upon uncemented shale, the shale will compact and the dam will settle. This settlement may be slight or it may be serious, depending upon the thickness of the shale, its degree of compaction, and the amount of additional load. The settlement will extend over a period of years, and its rate will depend upon the thickness of the shale and its permeability. The rate and extent of additional compaction can be determined from a "consolidation test" and from this and geological knowledge of the shale an approximate estimate of the amount and rate of settlement can be made. The fact that the shale may have formerly been subjected to a much greater load and may have had a higher degree of compaction does not prevent additional settlement when additional load is placed upon it. Several examples of concrete dams on shale will be described briefly to illustrate the points discussed.

Conchas Dam.—Built by the U. S. Army Engineers on the South Canadian River in eastern New Mexico, Conchas Dam is a good example of a dam underlain by both uncemented shale and sandstone.[23] The shale is a typical unlaminated, uncemented siltstone. Only two problems will be discussed herein. The north end of the dam, 90 ft high, rests upon a layer of sandstone 20 ft thick, underlain by 70 ft of shale which outcrops in the canyon wall. This part of the dam is supported by the shale, and the principal practical problems are shearing strength of the shale and settlement of the dam due to additional compaction of the shale under increased load. Analysis has shown that the highest stresses in the shale will be in the canyon wall at the heel and toe of the dam and that they will be less than the shearing resistance of the shale as determined by tests on specimens. These tests, however, could not take into consideration the effect of the numerous fractures in the shale; but this was checked by observations on the shale in the canyon walls where it is under load greater than that in the dam.

This shale was formerly under a greater load and was compacted more than at present, but it is still subject to greater compaction under increased load. It was computed that the load of the dam will cause a small, slow settlement which, under extreme conditions, might eventually be a few inches, but which will probably be much less. Since the higher, central part of the dam rests on sandstone below the shale, it will not be subject to this settlement and there will be differential movement between the parts of the dam which is provided for by a special joint.[24]

Tygart Dam.—Tygart Dam, 235 ft high, was completed in 1939 by the U. S. Army Engineers on the Tygart River in West Virginia. It is another high dam on sandstone and shales. The shales include both laminated and massive uncemented varieties, and partly cemented, laminated, and thick-bedded shales. The river-bed section of the dam rests upon a thick bed of sandstone, but the abutments are underlain by alternating beds of sandstone, sandy shale, thick-bedded carbonaceous shale, laminated shale, a massive uncemented shale, and thin beds of coal:

The principal problems were: Adequate foundations, prevention of sliding, prevention of disintegration during construction, and seepage. It was possible to have the foundations of each monolith on a bed of sandstone or sandy shale, thus avoiding the problem of foundations directly upon uncemented shale. Disintegration of the uncemented shales was a serious problem which was met principally by not excavating to the final lines until it was time to place concrete. Adequate resistance to sliding of the abutments was obtained by carrying the dam farther into the hillsides than would have been needed with stronger, more massive rocks. The bed of sandstone under the river section of the dam was underlain by a great thickness of shales with some thin coal seams. The shales were intersected by joint cracks and the coal seams were fractured. A grout curtain, 150 ft deep, was constructed and at first high grouting pressures were used; but the holes took excessive quantities of grout and it was impossible to grout some of them to refusal. It appeared possible that the high pressure was opening some of the bedding seams and the grout pressure was reduced, with the result that there was no further difficulty.

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[23] For a more complete discussion of the geology and foundation problems of Conchas Dam, see "Engineering Geology Problems at Conchas Dam, New Mexico," by Irving B. Crosby, *Proceedings*, Am. Soc. C. E., January, 1939, pp. 29–47.

[24] "Dam Building on Difficult Rock," *Engineering News-Record*, June 9, 1938, pp. 808–809.

Lake Lynn Dam.—Lake Lynn Dam, formerly called State Line Dam, on the Cheat River in West Virginia, near the Pennsylvania state line, has been in use since 1928. It is a good example of a concrete gravity dam, 100 ft high, resting almost entirely on shales, including the soft uncemented variety. It does not have the advantage of a thick bed of sandstone for the foundation of the high part of the dam as was the case with Conchas and Tygart dams, but, by means of special precautions, a satisfactory and safe dam was obtained. The high river section of the dam rests upon 10 to 15 ft of hard, gray, partly cemented, laminated shale. Beneath this stratum are beds of limestone, calcareous shale, coal, and uncemented shales. The principal problems were sliding and seepage. Sliding might take place in the laminated shales and seepage in the fractured coal. A very intensive investigation and analysis of conditions were made and they were met by the following procedures: The dam was keyed into the shale by a reinforced anchoring wall, extending 10 ft below the general foundation at the heel, and by a toe extending 7 ft below the general foundation. These throw the shearing stresses deep into the foundation shales and insure that sliding would involve shearing the shale across the bedding. The toe abuts against a mass of shale that is protected from erosion by a concrete apron. From the bottom of the anchoring wall a concrete cutoff extends down through the coal seam, and a grout curtain extends below that. The grout pressures used were generally 80 lb per sq in. and, although the rocks are laminated with flat bedding seams, there does not appear to have been any uplift. This dam site had several difficult conditions which were met successfully.

FAILURE OF DAMS ON SHALE

A number of dams on shale have failed by sliding and of these Ohio River Dam No. 26 is a typical example. The part which failed by sliding was of the Chanoine wicket type with a total height of about 20 ft. Apparently, it slid on a bedding plane in the shale about 0.2 ft below the base of the concrete.[28] The shale was laminated, soft, and poorly cemented, with a surface greasy to the touch. The toe of the dam did not abut against rock, and failure was caused by sliding on bedding seams along which resistance was low and had probably been further reduced by uplift. There does not appear to have been anything unusual about this failure and it is not necessary to suppose that there was an unpredictable system of joints.[25a] There is no record of any geological examination of this site having been made before the dam was built.

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[28] *Professional Memoirs*, U. S. Engr. Dept., Vol. 5, May–June, 1913, pp. 315–322.

Dams on Sandstone and Conglomerate

Sandstone is a rock composed of grains of rock or minerals of sand size. It grades downward into shale and upward into conglomerate. The grains are usually cemented together by calcareous, siliceous, ferruginous, or argillaceous cement but cement may be practically lacking. Calcareous sandstone grades into limestone, and argillaceous sandstone grades into shale. Sandstone may be thick or thin bedded, the thickness of individual beds varying from several feet down to a fraction of an inch. The thin-bedded varieties may have shaly characteristics. Sandstones are more or less porous and permeable. Conglomerate is consolidated gravel, the pebbles of which are cemented together.

The principal practical problems of masonry dams on sandstone are seepage, uplift, and sliding on bedding seams. Poorly cemented argillaceous sandstones or conglomerates may disintegrate with changed conditions and cause serious foundation problems and may render a site unfit for a masonry dam. Flat-bedded sandstones are especially susceptible to erosion at the toe of overfall dams and may require protection. Sandstones are intersected by joints and bedding seams and are also pervious, with the result that water can travel through pores, as well as through fractures, and hydrostatic pressure can diffuse throughout the rock. Therefore, problems of seepage and uplift are unusually important in sandstones.

There is great variation in the rate of weathering of sandstones due principally to the varying resistance of the different cements. Siliceous sandstones are the most resistant, and argillaceous sandstones the least so. The solution of the calcareous cement from a sandstone is believed to be generally too slow to be serious during the life of a dam, but research is desirable on this point. Shaly beds and partings weather faster than the more massive sandstones and this may require more extensive foundation excavation.

The crushing strength of sandstones and conglomerates varies greatly, and each formation must be tested separately. A series of unconfined compression tests on sandstones gave results varying from 4,920 to 33,350 lb per sq in.[26] A calcareous sandstone crushed at 18,810 lb per sq in. and a series of tests upon ferruginous sandstones gave results varying from 7,272 to 18,350 lb per sq in.[27] Some argillaceous sandstones and some of the sandstones with little or no cement are much weaker than those for which figures have been given. The argillaceous conglomerate from St. Francis Dam had a strength of only 523 lb per sq in., but the siliceous Roxbury conglomerate had a compressive strength of 17,360 lb per sq in.

[25a] "Effect of Minor Geologic Details on the Safety of Dams," by Charles Terzaghi, M. Am. Soc. C. E., *Technical Publications No. 215*, A. I. M. E., New York, 1929, p. 31.

[26] "Physical Properties of Typical American Rocks," by John H. Griffith, *Bulletin No. 131*, Iowa Eng. Experiment Station, 1937, pp. 10–11.

[27] "A Descriptive Catalogue of the Building Stones of Boston and Vicinity," by W. O. Crosby and G. F. Loughlin, *Technology Quarterly*, Vol. 17, 1904, pp. 165–185.

Sliding of a dam on sandstone takes place by shearing along bedding seams and is dependent upon the shearing resistance of the sandstone parallel to the bedding. The shearing resistance when wet is usually less than when dry, and alternate drying and wetting further reduces the shearing resistance of some sandstones, especially the shaly ones. The most important factor in regard to sliding is the presence and relation to the dam of shaly seams or open bedding cracks.

Conchas Dam.—Conchas Dam has already been described as an example of a dam on uncemented shale; but the highest part of the dam rests upon sandstone containing water under artesian pressure, and is an excellent example of dams on this rock. Since this sandstone is porous and permeable, and is intersected by joint cracks and bedding seams, the problems of seepage and uplift were important.[23]

Tygart Dam.—Tygart Dam has already been described as an example of a high dam upon shales, but the highest section rests upon a bed of sandstone. The layer of sandstone upon which the high section of the dam rests is thick bedded but is cut by joint cracks into irregular blocks. In order to make it more nearly a continuous layer "consolidation grouting" with shallow holes on 20-ft centers was resorted to over the entire base of the dam. The most serious problem was the prevention of sliding. The dam was bonded to the rough, irregular surface of the sandstone with the result that sliding would involve shearing across the bedding of this thick-bedded, strong rock, which is at least 14 ft thick. Since this sandstone is stronger than concrete, sliding is practically impossible as long as the bed of sandstone immediately downstream from the dam is kept intact, which is provided for by a stilling basin.

FAILURE OF DAMS ON SANDSTONE AND CONGLOMERATE

The greatest danger of failure of dams upon the siliceous, calcareous, or ferruginous sandstones is from sliding. These rocks are always more or less stratified, and sometimes the bedding seams are definite cracks. At Austin, Pa., a concrete gravity dam, 50 ft high on sandstone with shale bedding seams, failed by sliding. Water seeping under the dam probably caused uplift, and the dam slid on a shale bedding seam just below the foundations. No geological examination of the site had been made before the dam was built.

With dams upon argillaceous sandstone and conglomerate there is the added danger of weakness of the rock and the effect of water upon it. The St. Francis Dam was a good example of the failure of a dam on argillaceous conglomerate. The western part of the St. Francis Dam rested upon such a conglomerate, which consists of sand and pebbles of hard rock cemented with clay. It is essentially an uncemented mudstone containing pebbles, and has a very low compressive strength.[28] The reports made after the failure agreed in assigning this weak "conglomerate" as an important cause of failure.[29] This rock disintegrates when placed in water and illustrates the danger of such argillaceous sandstones or conglomerates in foundations. Since there was no other cement than clay, and since this softened when wet, the rock lost what strength it had in the dry condition and the dam failed. No geological investigation was made of the site before the dam was built.

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[28] "Essential Facts Concerning the Failure of the St. Francis Dam," by the late Louis C. Hill, Past-President, Am. Soc. C. E., H. W. Dennis, and F. H. Fowler, Members, Am. Soc. C. E., Report of Committee of Board of Direction, Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., Vol. LV, October, 1929, p. 2156.

[29] *Loc. cit.*, pp. 2147–2163.

DAMS ON LIMESTONE AND OTHER SOLUBLE ROCKS

Limestone is the most common soluble rock; but in this class are also marble, dolomite, gypsum, and rock salt. Limestone is composed of grains or crystals of calcite, or of fragments of calcium carbonate, as with shell limestone or coquina. The rock dolomite is a magnesian limestone composed principally of the mineral dolomite. Marble is metamorphosed, crystalline limestone.

The principal practical problems of dams on limestone and other soluble stratified rocks are leakage and sliding on shaly bedding seams. Limestones are intersected by joints and bedding seams in much the same manner as sandstones are, and jointing is often developed in a very regular manner. Joints and bedding cracks, however, are of unique importance in limestones due to the fact that the rock is soluble and that water passing through the cracks enlarges them and may form extensive caverns. Limestone is more soluble than dolomite but is much less soluble than gypsum. Geologically, limestone dissolves rapidly, but from the human viewpoint its solution is very slow and it is not believed that it is sufficiently rapid to enlarge joints and other fractures seriously during the life of a dam. With gypsum the case is different and its solution is sufficiently rapid to be serious during the life of a dam. The formation of caves and underground drainage systems depends upon many factors such as: The frequency, regularity, continuity, and openness of joints and bedding seams; the degree of solubility of the particular limestone; the presence of more or less soluble layers; and the structure and the stage of physiographic development of the region. These and other factors must be studied carefully by the geologist in order that the nature and extent of solution channels can be determined. Subterranean drainage systems have divides which may or may not coincide with surface drainage divides. Underground drainage divides must be located because their relation to the dam is of great importance, and study of ground-water levels is of assistance in this problem. Although limestone presents many serious foundation difficulties and uncertainties, many successful dams have been built upon it. Many attempts have been made, with varying degrees of success, to make cavernous limestone foundations tight by grouting. This subject has been treated adequately in previous papers.[30]

Limestones vary greatly in strength, a dense variety having a crushing strength of more than 40,000 lb per sq in. and coquina, shell limestone, a crushing strength of 220 lb per sq in. Many limestones crush at 10,000 to 20,000 lb per sq in. Sliding of dams on limestone will involve shearing of the limestone if the dam has been properly bonded to the rock. Most limestones have a high shearing resistance transverse to the bedding, and sliding is possible only along bedding seams. Therefore, if the foundation in such rocks is so designed that sliding would involve shearing across the bedding, danger of sliding is generally eliminated. If the limestone is thin bedded or has shaly partings, the possibility of sliding is increased.

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[30] "Improving Foundation Rock for Dams," by James B. Hays, M. Am. Soc. C. E., *Civil Engineering*, May, 1939, pp. 309–312; also "Corewall Grouting at Chickamauga Dam," by James B. Hays, *Engineering News-Record*, April 27, 1939, pp. 59–60.

Wilson Dam.—Situated at Muscle Shoals, Ala., on the Tennessee River, Wilson Dam is a good example of a successful dam built on limestone. The problems involved prevention of seepage and leakage around the dam through caverns and prevention of sliding and erosion below the dam. A thorough geological investigation was made. Under the river are alternating thick beds of chert and limestone. Above these, and forming most of the abutments, is thin-bedded, interlaminated chert and limestone. The beds dip gently to the southwest—that is, downstream and toward the left bank.

Numerous small caves occur in the south bluff; there are larger caves in the vicinity and large springs at Tuscumbia, 6 miles downstream. There had been some fear that these springs were connected with the river and that there might be serious leakage when the reservoir was full. A very thorough study was made of caves, joints, and springs in the vicinity. It was shown that the large springs were the outlets of the underground drainage system of a large area south of the dam, that there was an area of high ground water between this area and the dam, and that consequently there could be no connection between the springs and the river at or above the dam. It was also shown that the rock in the river bed could be made tight by reasonable grouting and that the only place where special treatment was necessary was at the south abutment. Two tunnels were driven into this rock and all open seams were cleaned out. Extensive grouting was done, and the tunnels were filled with concrete. The rocks under the river were susceptible of uplift from excessive grout pressure; but this was avoided by restricting the pressure to 30 lb per sq in.

Hales Bar Dam.—Hales Bar Dam on the Tennessee River is an excellent example of the difficulties that may arise when a dam is built on limestone without adequate investigation. In so far as can be learned, no geological investigation was made of the site before the dam was built, but several investigations have been made since to learn what could be done to remedy conditions. The dam is on limestone which is cavernous, and great difficulty was encountered in unwatering the cofferdams. Much grouting was done and compressed air caissons were used.[31] Since completion there have been very serious leaks under the dam which could not be stopped by any method until asphalt grouting was tried. This was partly successful and reduced the leakage. It is reported that $10,000,000 was spent in attempting to control the leakage. It is certain that if an adequate geological investigation had been made before the dam was built these difficulties could have been foreseen and it is probable that a better site could have been found.

FAILURE OF DAMS ON LIMESTONE

A recent example of serious leakage under a dam on limestone, where investigation of the condition of the limestone before construction had been inadequate, is the Ontelaunee Dam in Pennsylvania. Leakage developed under the core wall of this earth dam shortly after the reservoir was filled, and the embankment was endangered. The reservoir was drained and the condition was finally remedied by additional grouting.[32]

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[31] "The Power Development at Hales Bar," by J. A. Switzer, *Resources of Tennessee*, Vol. 2, No. 3, Nashville, 1922, pp. 86–99.

Some twenty years earlier, part of the masonry wall forming a reservoir at Nashville, Tenn., failed by sliding. The wall rested upon a thin-bedded, flat-lying limestone in which were bedding seams and thin strata of shale. The dam slid on a shale bedding seam 4 ft below the base of the concrete, and 4 ft of rock adhered to the concrete and slid with it. No geological examination of the site had been made before the dam was built.

The Austin Dam on the Colorado River in Texas is a well-known example of a dam which failed by sliding. The limestone has porous beds and shaly seams, and sliding occurred on one of these seams below the base of the concrete, a layer of rock sliding with the dam. The rock at the toe had been removed by erosion, and there was some undermining of the toe of the dam. The dam could slide on a bedding seam, therefore, without shearing across the bedding. No record has been found of any geological investigation having been made before the dam was built.

DAMS ON GYPSUM

Dams on gypsum present even more serious problems than dams on limestone because of the much greater rate of solution of gypsum. There have been a number of cases of very serious leakage under or around dams on gypsum, but there are also some reasonably tight reservoirs in gypsum. If there are no joint cracks or other fractures or openings for water to start through, gypsum may dissolve about the shores of the reservoir without causing any leakage. Apparently, such was the case with the Avalon Reservoir and Willow Lake in New Mexico. McMillan Reservoir, near the Avalon Reservoir, leaked badly, as did also the Honda Reservoir, also in New Mexico. In some cases projected dams on gypsum were abandoned when the conditions were disclosed by geological investigation.

DAMS AND RESERVOIRS ON BASALT AND OTHER LAVAS

Basalt and other lavas cover vast areas in the northwestern part of the United States and in other parts of the world, and therefore many dams have been built upon them. Since some of these lava regions are also semiarid regions, the need of storage reservoirs in them is great. Basalt includes the dark-colored lavas as differentiated from the felsites, or light-colored lavas. Basalt may be a massive, dense rock, it may be intensely fractured, or it may be porous. The felsites are less likely to be intensely fractured, but they may be extremely porous.

These lavas present some of the most difficult problems in dam geology, principally because of the fact that they are generally highly fractured. The problems, therefore, are principally concerned with loss of water, and this may not necessarily occur at the dam site. Sometimes the rocks at the dam site are sufficiently tight, or can be made so, but serious leakage may occur from other parts of the reservoir basin. There are cases in which dams that have been entirely satisfactory in themselves have been rendered useless by the fact that severe leaks elsewhere in the reservoir basin made it impossible to store water, and in some cases impossible even to fill the reservoir. The most important problem of dams on basalt is likely to be tightness of the reservoir rather than safety of the dam.

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[32] "Caverns Under Dam Corewall Set a Nice Repair Problem," by Farley Gannett, M. Am. Soc. C. E., *Engineering News-Record*, April 2, 1936, pp. 492–494.

Basalts may be more intensely fractured than most other rocks or they may be massive and relatively free from close jointing. Basalts which formed as thin flows and cooled quickly may have the typical columnar jointing which divides the rock into columns, sometimes hexagonal, of several inches to a few feet in diameter. Each column is surrounded by cracks and since these cracks were formed by shrinkage of the cooling mass they are less likely to be tight at depth than other joint cracks. As a result some basalts permit free movement of water.

Lava tunnels in basalts are a unique type of open passage which is of extreme importance in relation to dam sites and reservoir basins. These are natural tunnels in the lava which may extend for miles and may be 30 ft or more in diameter. They may be open or partly or entirely filled with sand which has washed into them. Those which are known were discovered where the roofs had fallen in, and there are doubtless many which have never been discovered. Obviously, one of these tunnels underlying a reservoir and leading outside it could cause tremendous leakage. Their discovery by drilling only is not practicable in a large reservoir basin and the best method for locating them is by a thorough geological and ground-water investigation with drilling at suspicious points.

Because of the close jointing and the possibility of lava tunnels, basalts are among the most leaky and treacherous rocks with which the dam builder has to deal. Ground-water studies, therefore, are especially important for reservoirs in basalt. If it is found that the ground-water table is far below the reservoir basin, or if there is a deep canyon nearby, the probability is great that there will be serious leakage from the reservoir. However, if it is found that the ground-water table is high and rises from the reservoir basin, it is probable that a dam can be built safely and that the reservoir will be sufficiently tight. In general, basalts and other lavas are very strong rocks, but their strength is determined more by the amount of fracturing than by their mineralogical composition. Therefore, any tests upon specimens must be interpreted in the light of a thorough geological investigation of their structure.

There are many successful dams on lavas and some which have been failures, due to the fact that the reservoirs would not hold water. A few examples will illustrate some of the problems. The two examples of failure happen to be reservoirs formed by earth-fill dams, but since the leaks were not at the dam site they had nothing to do with the type of dam and would have occurred with masonry dams.

American Falls Dam.—At American Falls on the Snake River in Idaho, a concrete gravity dam 87 ft high was completed by the Bureau of Reclamation in 1927. The dam is founded on a bed of relatively massive columnar basalt which is underlain by a bed of obsidian (volcanic glass) about 23 ft thick. This in turn is underlain by a bed of tuff. The dam site is intersected at the western end by an old fault. The dam is near the crest of the falls, a position which was required by the topography but which tends to increase seepage, due to the possibility of short-circuiting under the dam to the river below the falls. The reservoir basin which is approximately 25 miles long is generally, but not completely, blanketed by relatively impervious, sedimentary lake deposits, clays, and fine sands. Although there has been considerable seepage from the reservoir, it has not interfered with its economically successful use for irrigation storage. The blanket of impervious lake beds on the floor of the reservoir is at least partly responsible for its success.

Jerome Reservoir.—The Jerome Reservoir on the Snake River lava plain about 8 miles northeast of Jerome, Idaho, was formed by two earth-fill dams and was fed by a canal. A layer of volcanic ash covered the bottom of the reservoir but was underlain by fractured basalt. The ground-water level was very low, which should have been a warning. When it was attempted to fill the reservoir, water broke through the layer of ash into the underlying basalt, forming funnel-shaped sinks in the ash. A flow of 2,000 cu ft per sec would not fill the reservoir, which was then abandoned. The contrast between this reservoir and the successful American Falls Reservoir is due to the contrasting geological conditions. Jerome Reservoir was in a dry valley where the ground-water level is very low, whereas American Falls Reservoir is in a river valley which had once been occupied by a lake, and which had a nearly complete blanket of impervious sediments on its floor; also, the ground-water level was higher.

Tumalo Reservoir.—Tumalo Reservoir on the eastern slope of the Cascade Range, in Oregon, is another example of failure similar to the Jerome Reservoir. It was formed by an earth-fill dam across a small canyon. The bottom of the reservoir was covered with volcanic ash underlain by fractured basalt. The water seeped through the ash, carried it into the fissures, and formed open sinks. This reservoir was abandoned.

DAMS ON GRANITE AND OTHER INTRUSIVE IGNEOUS ROCKS

The practical problems of dams on the granites and on the other intrusive igneous rocks are similar. These rocks include the true granites, syenite, diorites, and gabbros. They were all hard, strong rocks when formed, but they may have been so altered that their present characteristics are unfavorable for foundations. They may be among the best foundation rocks, but they may be in a very unsatisfactory condition due to fracturing, alteration, or other causes. The practical problems of dams on the granitic rocks are concerned with faults, shear zones, weathering and disintegration, and zones of alteration.

Like all rocks granites may be cut by faults and shear zones, and they are usually intersected by rather regular systems of joints. The joints may be somewhat open near the surface and may provide possible paths of leakage, necessitating grouting. Since these rocks are insoluble, however, water passing through cracks cannot enlarge them and open channels cannot occur. The granitic rocks are all susceptible to weathering, and in general the coarse-grained rocks are less resistant than the fine-grained ones. Sometimes the feldspar in the granite kaolinizes and destroys the bond between the crystals, with the result that an apparently sound ledge breaks down easily into gravel. Sometimes in a biotite granite the biotite hydrates and swells, causing the rock to disintegrate. In warm, humid regions, weathering may penetrate to great depths.

Many successful dams have been built upon granite and similar igneous rocks. The Grand Coulee Dam and Diablo Dam in Washington, the O'Shaughnessy Dam and Morris Dam in California, the Arrowrock Dam in Idaho, and the Buchanan Dam in Texas may be mentioned.

Rapide Blanc Dam.—The Rapide Blanc Dam on the St. Maurice River in Quebec, Canada, a concrete gravity dam 145 ft high, had an almost ideal site on gneissic granite; yet the first site examined, about half a mile upstream, was abandoned because of the badly fractured condition of the rock. Sheet jointing dipped toward the river in the right bank, and the natural tendency for blocks to move down the joint planes had been accentuated by action on a nearby fault. The present site was then investigated and it was found that the sheet jointing was not well developed, that joints did not dip toward the river, and that they had not been affected by faulting. Sound granite was at or near the surface everywhere, and a better dam site is difficult to find.

Forks Dam Site, San Gabriel Canyon.—The Forks Dam site in San Gabriel Canyon, California, illustrates the extreme changes produced by faulting, crushing, alteration, and decay in originally strong granitic rocks. The site is at the junction of two fault zones, and the rock is cut in every direction by innumerable faults and crush zones that divide it into blocks of varying size, which are themselves fractured. In addition to the crushed and fractured condition of the rock, much of it is deeply decayed and disintegrated and alteration extends to depth along the fault zones.[33]

Dams on Schist and Slate

Schist and slate are metamorphic rocks which have distinctive physical characteristics in regard to dam foundations. The principal characteristics of these rocks from the foundation viewpoint are foliation and cleavage. The schists have a foliation which may be wavy. They split easily with the foliation but break with difficulty across it. The cleavage of slate is smoother and more regular and may divide the rock into thin sheets.

The principal practical problems of dams on slate or schist are sliding and stability of abutments. Sliding might take place on either the cleavage planes of slate or the foliation planes of schist. Mica schist probably has the lowest resistance to sliding of any of the schists. Cleavage planes are usually not actual fractures but they are planes of weakness. Schists and slates are also intersected by joints but these are often not so well developed as in sandstones and limestones. Neither slate nor schist disintegrates as do the uncemented shales. Some schists are very resistant to weathering, whereas others weather easily and deeply.

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[33] "Serious Defects in Foundation Rock Stop Work on San Gabriel Dam Project," *Engineering News-Record*, October 31, 1929, pp. 699–701; also "State Denies Permission to Build San Gabriel Dam as Proposed," *Engineering News-Record*, December 5, 1929, p. 895.

Schists vary greatly in strength and each variety must be tested. Some slates are moderately strong rocks as shown by compression tests on three slates which had a strength of 17,700 to 19,199 lb per sq in. transverse to the cleavage. The shearing resistance of both slate and schist is much greater across the cleavage than parallel to the cleavage. The relation of the cleavage to the dam, therefore, is very important.

Fifteen Mile Falls Dam.—A good example of a successful high dam on schist is the Fifteen Mile Falls Dam on the Connecticut River in northern New Hampshire. This is a concrete, gravity dam, 175 ft high, completed in 1930, resting upon schist which is in part slaty. A very thorough geological investigation was made and was continued during the construction period.[34]

Prettyboy Dam.—Some of the problems due to weathering of schist and to the difficulties of excavation in some schists were illustrated at the Prettyboy Dam. This is a gravity, concrete dam, built for the water supply of Baltimore, Md. In the lower part of the west abutment sound schist was about 8 ft below the surface, but in the upper and westernmost part of this abutment sound rock was more than 100 ft deeper than expected, which necessitated greatly increased excavation. The weathered rock had been dropped down behind the fresh rock, by a fault, giving a deceptive appearance to the face of the abutment. Even where the rock was sound, the ease with which shooting split the rock along the planes of schistosity made it difficult to obtain a good bottom for the foundation excavation and necessitated much additional excavation. This difficulty was finally obviated by the use of wire saws and the avoidance of explosives.[35]

St. Francis Dam.—In addition to the possibility of sliding on cleavage planes, there exists the danger of injury to the dam from instability of the abutments if the strike of the schist parallels the valley and the dip is parallel to, or flatter than, the valley wall. This was illustrated at the east abutment of the St. Francis Dam. The east wall of the canyon is formed of mica schist with the planes of schistosity dipping steeply to the northwest parallel to the wall of the canyon. Any undermining of the canyon wall would remove the support of masses of schist above, which would avalanche into the canyon. Most of those who investigated the failure of the dam concluded that the schist was undermined by the rush of water from the break in the western part of the dam and that large slides then occurred; but one commission concluded that slides in the schist were largely responsible for the failure of the dam.[29] In any case, such an unstable abutment is most undesirable, and such conditions are not rare since the slope of canyon walls is frequently determined by planes of schistosity or cleavage, or even by the bedding of sedimentary rocks.

CONCLUSIONS

From the preceding examples and discussions, the many difficulties that may arise from not meeting the geological conditions at a dam site are apparent.

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[34] "Geology of Fifteen Mile Falls Development," by Irving B. Crosby, *Civil Engineering*, January, 1934, pp. 21–24.

[35] "Redesign and Construction of Prettyboy Dam," *Engineering News-Record*, July 20, 1933, pp. 63–67.

The many circumstances which may reduce the value of a dam, due to leakage or other defects, or may greatly increase the costs, unless properly provided for, have been discussed and it has been emphasized that these may be predicted in advance. When they have been foreseen, and when the engineer has received full information about the foundation conditions, he can meet them, howsoever bad, unless he finds it uneconomic to do so. The same system of investigation which has revealed unfavorable natural conditions can usually find an alternate, more favorable site. Dams may fail from many causes, due to many types of defects in the foundation rocks; but there is nothing mysterious about these failures. They are all due to definite geological conditions which can be discovered in advance if scientific knowledge and experience are given adequate opportunity.

CONCRETE CONTROL

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SYNOPSIS

Concrete suitable for use in dam construction must have the following characteristics: Strength sufficient to carry the loads and provide a required factor of safety; weight, in the case of gravity dams, to provide safety against sliding and overturning; durability to insure against weathering and erosion; impermeability to prevent water percolation and solution of cementing material; and continuity in order that the structure may act according to assumptions of design. These essentials are controlled by various factors which may be grouped under two headings: First, materials from which the concrete is made, and second, methods of using the materials. Materials available and methods for using them scarcely can be considered separately since one is often dependent on the other, and the design of concrete therefore requires consideration of materials and construction procedure as well as type of structure in which it is to be used. It follows, then, that problems of selecting materials and determining construction procedures and schedules should be considered as problems of design.

Although knowledge concerning properties and behavior of concrete under various influences has been enormously increased during the last few years, an urgent need for additional investigations, mainly in the field of mass concrete, in which the greatest strides have recently been made, still exists. Application of knowledge gained from laboratory tests and from field observations has aided greatly in bringing the quality of dam structures to its present state. Future progress depends on utilizing available information to the fullest extent as well as on investigating further the many phases of concrete behavior still not well understood.

This paper is an attempt to describe, very briefly, the present state of progress in concrete manufacture and control as applied to construction of dams, with some mention of factors that may be of importance to future developments. As the scope of this paper will permit only a small amount of detailed discussion, a list of references is included in the Appendix for the benefit of those who may wish to study some of the subjects mentioned.

MATERIALS

Cement.—In addition to having properties assuring strength, durability, and impermeability of concrete, cement for large dams should have a low rate of heat evolution during hardening and a low total heat liberation. Search for such a cement has led to the development of low-heat portland cement and portland-puzzolan cements of various types. Extensive investigations before and during construction of Boulder Dam were largely concentrated on heat of hydration of cement because it was recognized that temperature effects were largely responsible for cracking in large masses of concrete. Out of these investigations and further studies on behavior of several concrete dams came a realization of the importance of another property of concrete—that of its ability to withstand temperature strains without cracking. Recent tests indicate that success of the low-heat cement in reducing cracking may be due as much to this property as to its favorable characteristics of heat liberation.

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[36] Research Engr., Portland Cement Assn., Chicago, Ill.

Following the development of low-heat cement and its use in Morris Dam, in California, and Boulder Dam, on the Colorado River, a modified portland cement came into general use for dam construction. This cement has a more closely controlled chemical composition and physical properties than the normal portland cement, and its heat of hydration lies about midway between the normal and low-heat types. It has part of the advantages of the low-heat cement without its disadvantages, and for a time it appeared that the modified cement best filled the general requirements of dam construction. Modified portland cement has been used successfully in many large dams and is still considered by many to have distinct advantages for some types of dam construction.

At the present time (1940) the use of low-heat portland cement for large dams is rapidly increasing and as problems attending its use are overcome, low-heat cement may be expected to displace the other types except in locations where portland-puzzolan cement may be more economical (or in extremely cold climates where construction difficulties may make its use impractical). Usual specifications for the three types of portland cements are compared in Table 10.

Portland-puzzolan cements present individual problems and scarcely can be considered as a class at the present time because of the variety of puzzolanic materials that may be used. This type of cement is essentially a mixture of portland cement and some form of active silica. Present thought is in the direction of intergrinding portland cement clinker of about "modified" composition with suitable puzzolanic material to form an intimate mixture of greater uniformity than could be obtained by batching the materials ground separately and depending upon the less thorough mixing during the manufacture of concrete. Portland-puzzolan cement has been used successfully in Bonneville Dam, in Oregon and Washington, and undoubtedly more will be heard of it in the future.

No definite statement of relative durability of concretes containing these types of cement can be made. It is generally agreed that concrete containing low-heat cement which has not been properly protected and cured during its early hardening stages is inferior in resistance to weathering to concrete containing normal or modified portland cement of equal curing history. Durability tests made at early ages are generally unfavorable to low-heat cement because of its slow rate of hardening. Freezing and thawing durability tests on properly cured concrete specimens at ages of approximately one year are not conclusive in establishing a definite superiority of any one type of cement, variations between brands of cement generally being greater than variations between types. Freezing and thawing tests have shown conclusively the necessity for proper protection and extended curing of low-heat cement concrete which is to be exposed to weathering.

TABLE 10.—COMPARATIVE SPECIFICATIONS FOR THREE TYPES OF PORTLAND CEMENTS

Item	Type of cement (1)	Ignition loss[a] (2)	COMPOUND COMPOSITION[b]		PHYSICAL PROPERTIES[c]		
			Tricalcium aluminate (3)	Tricalcium silicate (4)	Fineness (5)	Strengths[h]	
						7-day (6)	32-day (7)
1	Normal portland	4.0	....	....	....[d]	275[f]	350[f]
2	Modified	3.0	8	35 to 55	1,600 to 2,200[e]	1,500[g]	2,500[g]
3	Low heat	3.0	7	35	1,700 to 2,300[e]	1,000[g]	2,000[g]

[a] *Chemical Analysis.*—Maximum percentages; other items of chemical analysis, common to all three items, are: Insoluble residue, 0.85% maximum; sulfuric anhydride, 2% maximum; magnesia, 5% maximum; and ratio of iron oxide to aluminum oxide, 1.5% maximum (except item 1).

[b] *Compound Composition.*—Maximum percentages; in addition to values in Cols. 3 and 4, the following applies to item 3 only: Dicalcium silicate, 40% to 65%; and tetracalcium alumina ferrite, 20%.

[c] *Physical Properties.*—Standard steam soundness tests on neat cement pats showed each item to be sound. The initial set (using a Gillmore needle) occurred in 1 hr (minimum), and the final set occurred in 10 hr (maximum). [d] Measured in terms of the percentage passing or retained on a 200-mesh screen. [e] Measured in square centimeters per gram (by Wagner turbidimeter). [f] Minimum tensile strength briquets. [g] Compressive strength of 2-in. mortar cubes. [h] In pounds per square inch.

Fineness to which a cement is ground has been shown to have a marked effect on physical properties of fresh concrete in which it is used. It also affects rate of hardening and heat liberation. Coarse cement has a rapid rate of settlement in water and when used in concrete permits the movement of mixing water vertically through the concrete, producing the phenomenon of bleeding. Surface effects, such as sand streaking and water and laitance accumulations at the tops of concrete lifts, are common results of bleeding. Internal effects of bleeding are not well understood. For reasons which are still somewhat in doubt, low-heat cement seems to have a greater tendency toward bleeding than normal or modified cements of equal fineness. Therefore, somewhat finer grinding of low-heat cement is desirable. Sufficiently accurate control of fineness has been made possible by the development of several devices for determining approximately the surface area of cements. Finely ground cements harden and produce heat more rapidly than coarser cements because of the greater area of cement exposed for hydration. For mass concrete the high rate of heat liberation is considered undesirable, and modified cements are usually limited to about 2,100 sq cm per g (by turbidimeter) and normal portland to a somewhat lower value for such use. Experience indicates that for concrete mixes of 1 bbl of cement or less per cu yd, fineness of low-heat cement could be at least 2,200 sq cm per g with advantages of decreased bleeding and increased workability outweighing the disadvantage of increased heat of hydration. It is to be expected that bleeding and other characteristics of cement may be governed by its particle size distribution as well as by its specific surface.

Aggregate.—Ordinarily aggregate for concrete dam construction must come from the cheapest suitable source—in most cases the source nearest the dam site. As dams are usually built in outlying districts, it often happens that the available aggregate has no record in concrete upon which to base conclusions as to its suitability for the intended purpose. It then becomes necessary to determine by experimental means the various properties of the material which might affect its behavior when used in concrete. In the past the tendency has been to rely on visual inspection and the personal opinion and judgment of the engineer. In many cases much may be learned by careful observation of behavior of the material in outcroppings or exposed locations of the deposit, and evidence gained in this manner should in no way be minimized. However, present practice is to augment such information greatly, particularly in doubtful cases, with results obtained by subjecting the material to various durability tests, microscopic examinations, and chemical and other tests in order to furnish the engineer with more complete data on the material with which he has to deal. An unknown aggregate proposed for use in an important dam might be investigated by applying tests as follows: (1) Chemical analysis, (2) petrographic analysis, (3) absorption, (4) specific gravity, (5) abrasion, (6) sodium sulfate or magnesium sulfate test for soundness, (7) strength of the aggregate in compression, (8) strength of mortar and concrete containing the aggregate, (9) freezing and thawing tests on aggregate and on concrete containing the aggregate, (10) expansion due to temperature change, (11) expansion due to moisture change, and (12) thermal properties. Satisfactory performance in these tests and no unsatisfactory indications in the field investigations would practically assure that an aggregate would give good service in a concrete structure. Conflicting indications would be cause for caution or further investigation, if not rejection, of the aggregate.

Concrete aggregates may be of two general types—natural or manufactured. Suitable natural aggregates, usually siliceous in composition, are found in river or glacial deposits at some distance from their point of origin. Weathering and erosion have eliminated unstable material, leaving sound particles with rounded corners which are well suited to use as concrete aggregates. When such a deposit of reasonable particle size distribution is available, the problem of aggregate supply is solved. In areas where deposits may contain appreciable quantities of unsound chert, shale, soft sandstone, or other undesirable material, the problem becomes involved and the manufacturing of aggregates from native rock may have to be considered.

Manufactured aggregate crushed from suitable deposits of various types of rock has been used successfully for some time as coarse aggregate for concrete. Use of crushed sand has not always been entirely satisfactory, leading in many localities to rejection of manufactured sand in favor of natural sand, even at a considerable premium in cost. The unsatisfactory performance of crushed sand in concrete has undoubtedly been due to unsound rock in some cases, but probably has been due more often to poor particle shape and poor particle size distribution and consequent high water requirement of the concrete in which it has been used. Both coarse and fine crushed aggregates are being used at the present time in producing concrete which is reasonably certain to compare favorably with concrete containing the best of natural aggregates. Fig. 7 shows the effect of methods of manufacture on the particle shape of aggregate. The aggregate shown in Figs. 7(*a*) and 7(*b*) requires appreciably less cement than that shown in Figs. 7(*c*) and 7(*d*).

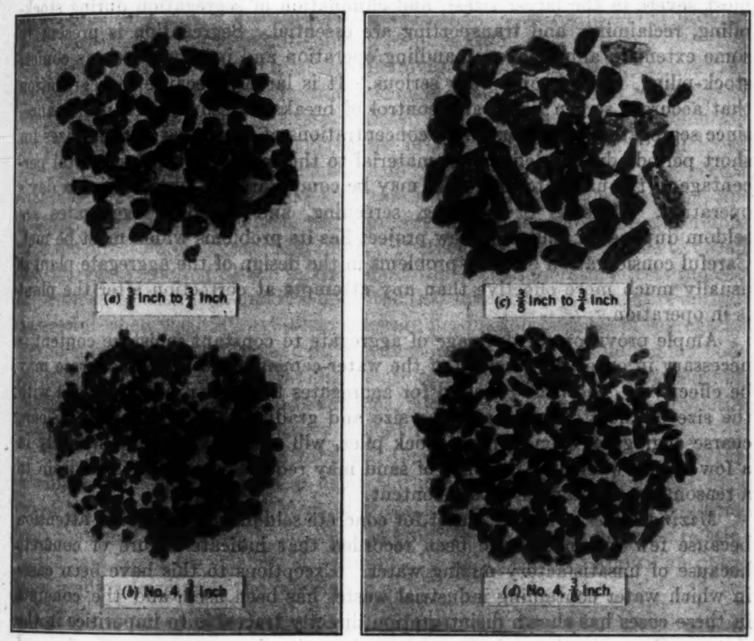

Fig. 7.—Particle Shapes Produced by Different Manufacturing Processes

Regardless of which type of aggregate is used, it is essential that processing plants and handling methods and equipment be such that the combined aggregate at the mixing plant will be clean and of a uniform grading which may be controlled within limits appreciably affecting the quality of concrete. In order to permit accurate control of mixing water, aggregates must be of uniform moisture content when delivered to the mixing plant. Usual best practice is to wash the coarse aggregate, either before or after screening to size, and to wash and classify sand by means of wet classifiers. For mass structures in which aggregate sizes of 6 in. or larger are used, four sizes of coarse aggregate are usually required and either one or two sizes of sand in order to satisfy minimum permissible variations in combined aggregate grading. Aggregates for mass concrete are usually separated into the following nominal size groups:

Size	Group
3 in. to 6 in.	Cobbles
1½ in. to 3 in.	Coarse
⅝ or ¾ in. to 1½ in.	Medium
No. 4-mesh to ⅝ or ¾ in.	Fine
Smaller than No. 4-mesh	Sand

Accurate sizing of the different aggregates, control of breakage (usually most severe in the larger sizes), and elimination of segregation during stock-piling, reclaiming, and transporting are essential. Segregation is present to some extent in almost every handling operation and in some, such as conical stock-piling, may become very serious. It is largely because of segregation that accurate screen sizing and control of breakage assume such importance, since segregation may cause high concentrations of unwanted particle sizes for short periods during delivery of material to the mixers, even if the total percentage of the unwanted material may be comparatively small during a day's operation. Methods of crushing, screening, and handling aggregates are seldom duplicated, and each new project has its problems which must be met. Careful consideration of these problems in the design of the aggregate plant is usually much more effective than any attempts at correction after the plant is in operation.

Ample provision for drainage of aggregate to constant moisture content is necessary in order that control of the water-cement ratio of the concrete may be effective. The time required for aggregates to drain properly varies with the size of the stock pile and the size and grading of the material. Clean, coarse aggregate, even in large stock piles, will normally drain sufficiently in a few hours, whereas large piles of sand may require several days to drain to a reasonably constant moisture content.

Mixing Water.—Mixing water for concrete seldom receives much attention because few instances have been recorded that indicate failure of concrete because of unsatisfactory mixing water. Exceptions to this have been cases in which water containing industrial wastes has been used, and the concrete in these cases has shown disintegration directly traceable to impurities in the water. Concentrations of silt and organic matter in most streams or rivers of low industrial waste pollution are usually insufficient to affect properties of concrete noticeably. Excessive quantities of silt should be removed by clarifiers and organic material by sand filters or other means. Water containing chlorides, sulfates, or other soluble materials in unusual amounts should be viewed with suspicion. Mixing water for any important structure should be tested for impurities of harmful nature which approach dangerous limits. Tests should include use of the water in mortar or concrete test specimens.

CONCRETE CONTROL

Proportioning.—In the absence of actual test data the water-cement ratio offers the most acceptable measure of potential concrete quality. This is true with possible rare exceptions, because, with proper curing and attention to the other factors of concrete quality, the water-cement ratio is an accurate measure of the quality of the weakest, least durable portion of portland cement concrete —the cement paste. The presence of some water is necessary to the hardening process of the cement paste (formation of cement gel), and the presence of a much greater quantity of water is necessary in order that the concrete may be consolidated in place with a minimum of void spaces between particles of aggregate and cement. The fact that use of more cement in a concrete mix raises the quality of the cement paste (water content of concrete at fixed workability changes little with cement content), and at the same time adds to the quantity of the least stable ingredient, presents an involved problem for which there is not a definite answer at present.

Designing of concrete mixtures involves the combination of available materials in such a way as to produce concrete of predetermined quality most economically. Normally this resolves itself into determining the combination, or possibly combinations, of aggregates of different particle size which will require the least amount of cement paste of fixed water ratio to give plastic, workable concrete under actual field conditions of mixing, transporting, and placing. The first step in laboratory investigation of concrete mixes, therefore, is in the direction of aggregate grading. It is seldom, if ever, possible to obtain the aggregate grading at the mixing plant which the laboratory tests indicate to be the most desirable, and it is not often that laboratory gradings are entirely successful when used in large-scale concrete production because of segregation during mixing and handling and other deficiencies in plant or handling methods. In spite of such limitations, intelligent use of laboratory information, suitably modified to approximate field conditions, is of inestimable value in predicting material requirements and quality of concrete.

Many methods have been proposed for determining theoretical aggregate gradings, each with the idea of using the minimum amount of cement for a given water-cement ratio. Most of these methods approximate a condition such that quantities of each particle size group bear a fixed volumetric relation to the next larger (or smaller) size group. Probably the best known and most used grading formula is the familiar

$$p = \left(\frac{d}{D}\right)^n \qquad (4)$$

in which p is the proportion passing a given screen size d; D is the maximum size of aggregate; and n is a factor roughly measuring harshness of the mix. Plotting the percentage passing against screen size on double logarithmic paper makes a very convenient method of handling this equation. The slope of the grading curve (then a straight line) is measured by n. The use of theoretical gradings must be accompanied by judgment on the part of the engineer. It is seldom if ever possible to calculate an aggregate grading and proportion a concrete mix without experimental data or actual trial batches of concrete as a guide. Too close adherence to theoretical gradings usually leads to undersanded and harsh concrete mixtures.

Batching and Mixing.—Concrete batching and mixing equipment has developed rapidly during the last few years. Development has been largely in the direction of increased capacity because of demands of the many large construction projects. Batching equipment has improved notably in accuracy and precision of operation, as well as in handling capacity; but concrete mixers, with the exception of those on a few well-known projects, have not received the attention necessary to force the much needed development in this class of construction equipment.

Automatic weigh batching equipment is rapidly displacing manually operated batchers on large concrete projects because of its greater uniformity and precision in operation and speed in batching. Manual batching may be of equal or even greater precision over short periods of carefully supervised operation; but fatigue of the operator and pressure for high rates of concrete production often cause erratic batching. Weigh batching of water is generally preferred to volumetric batching, particularly in plants where automatic recording of quantities of materials is required. The better class of batching equipment has been developed to a point where it is superior to most other important items of plant equipment and operating procedures that affect quality of concrete.

Charging of mixers and the mixing operation are beginning to receive some careful attention. The two are considered together because investigations have shown that charging sequence of ingredients of the concrete batch may materially affect the quality of concrete produced in some mixers during the normal mixing times required. Heretofore about the only attention paid to charging sequence has been with the idea in mind of preventing building up of materials behind the mixer blades. It has been the practice for construction organizations to aid manufacturers in developing other items of construction equipment most essential to concrete production, handling, and placing; but until recently the development of concrete mixers has rested almost entirely with the manufacturers. Since it would scarcely be practicable for a manufacturer to do much experimenting with a large mixer, except in collaboration with some going construction project, the future progress in mixer design will depend largely upon such cooperation. An appreciable amount of significant experimental and development work has been conducted with marked improvement in mixer performance. Further progress may be expected in the future since serious attention has been directed to the problem.

The most important factor governing quality of concrete is control of mixing water. Modern batching equipment provides means for adding water in accurately determined amounts. This often gives a false sense of security to engineers who do not fully appreciate the difficulties involved in determining the quantities of free water carried by aggregates. Including sand, aggregates often contain more than one third of the total amount of mixing water required for the manufacture of concrete. So long as these moisture contents remain constant and are determined accurately, precise control of concrete mixing water is possible and full advantage of good batching equipment may be realized; but if the moisture contents vary widely, the value of excellence of batching equipment is greatly reduced. Development of consistency meters as indicators of mixing-water content offers promise where constant aggregate gradings are known to exist.

Transportation of Concrete.—Concrete should be transported quickly from mixer discharge to the point of deposit, and without segregation of materials. Segregation occurs in every handling operation to some extent, including discharging of mixers and dumping of concrete buckets; and it is important, therefore, that the number of additional handling operations be reduced to a minimum. With large aggregate sizes, best results have been obtained by moving the concrete, preferably batch by batch, from mixers to form with no storage between, thus assuring that each batch reaches the forms with the materials it should contain. Bottom dump concrete buckets, loaded directly from the mixers or from intermediate transfer devices handling single batches or multiples of batches not exceeding the bucket capacity, are necessary in this procedure. Concrete transfer systems using methods of moving concrete in thin streams with transfer points and storage hoppers are generally considered undesirable from a concrete quality standpoint because of the comparatively large degree of segregation that may take place and the loss in flexibility of plant operation. Such systems are used occasionally for economic reasons and may be justified in some instances; in general they should be avoided. For concrete containing smaller maximum-size aggregate, pumping offers a feasible method of transportation under suitable conditions.

Concrete Placing.—Modern construction practice requires that concrete shall be deposited approximately in its final position in the forms. For dam construction, bottom dump buckets handled by cranes or cableways have come into general use as best fitting the usual needs. Buckets of 3-cu-yd, or less, capacity are usually dumped manually, whereas larger buckets are often dumped by power in order to decrease danger to workmen and to save time. Both square and round buckets are used, square buckets generally with comparatively fast uncontrolled discharge for mass concrete, and round buckets mostly of the controllable discharge type for either mass concrete or for concrete to be placed in thin or reinforced sections.

In starting a lift of concrete in a dam, normal procedure is first to deposit a batch of sand-cement mortar which is wire-broomed thoroughly into the old concrete surface or rock foundation, leaving a layer perhaps ½ in. thick. Some question still remains concerning the proper stage of dampness of the old concrete surface at the time it is to receive the mortar. It is generally agreed that the old surface should not have been allowed to dry out before starting the following lift of concrete, but there is evidence that the surface should not be wet at the time mortar is deposited. Presence of even small pools of water over the surface is highly undesirable. Some authorities believe that the best bond is obtained when the surface has been allowed to dry to a natural-surface, dry condition just before concrete placement is started. To serve its intended purpose most effectively mortar must be of a consistency such that it may be brushed readily into small depressions of the old surface. The water ratio of the mortar may be that of the concrete that is to follow, or slightly lower.

Placement of concrete should follow immediately after spreading the mortar, covering the grouted area as soon as possible. In the placing of mass concrete, it is general practice to deposit the contents of each concrete bucket in adjacent piles, vibrating each pile by means of internal vibrators to a thickness which is readily penetrated to full depth by the vibrator and joining each pile in such a manner that individual deposits cannot be distinguished after the operation is completed. When small buckets of comparatively soft concrete are used, no particular difficulties are encountered; when the contents of buckets containing 6 to 8 cu yd of relatively dry concrete are being deposited, something of a problem is presented, requiring excellent vibrating equipment and close supervision in order to assure uniformity and complete compaction of the mass. Present tendencies are strongly in the direction of drier concrete handled in large buckets because of lowered placing costs. This is particularly true on dams using cableways for concrete placement. There is considerable question in the minds of many engineers about the desirability of the large buckets when viewed from the standpoint of concrete quality.

FIG. 8.—"NO SLUMP" CONCRETE BEING PLACED BY VIBRATION

There is little doubt that present tendencies toward drier concrete are generally justifiable on grounds either of quality or of costs. Attempts to use very dry concrete mixes are likely to be unsuccessful, however, unless highly efficient, well-serviced vibrating equipment is available and unless the difficulty of handling the dry concrete is thoroughly appreciated by those doing the work. Opinions concerning the advantages of different types and makes of vibrators vary greatly among construction men and engineers, but there is agreement on the value of internal vibration of concrete as opposed to surface vibration (see Fig. 8). For mass concrete placement, the choice of vibrators lies between electrically driven and compressed-air machines. When used in dry mass concrete, most vibrators require two men for continuous operation. Electrically driven vibrators operating at approximately synchronous speeds offer dependability to a high degree, but they are heavy and until recently could not be used at high vibrating speeds. Air driven vibrators generally have advantages of high speeds and light weight, but are dependent on a uniform air supply for satisfactory operation. Much controversy has taken place over high-amplitude, low-speed operation of vibrators versus low-amplitude, high-speed operation. At this time there is a definite lack of information on the subject, but it appears likely that for each type of concrete mix there is some combination of frequency and amplitude best suited to placement of that particular mix. In general, vibrating frequencies are being increased with beneficial results, although recent discussions on vibration of concrete, emphasizing the advantages of high frequencies, have led to a general disregard of the necessity for sufficient amplitude. Size and shape of the vibrating element also have marked effects on vibrator performance.

At considerable risk of starting a controversy, it seems appropriate to consider the problem of placing concrete in wet weather. This subject is neatly avoided in nearly all specifications for dam construction, although it is known by all careful observers of concrete that it is not possible to place the same quality of concrete in even a light rain that can be placed under favorable dry-weather conditions. The most serious danger exists at the beginning of a concrete lift where mortar covering the old concrete may be diluted with rain water to such an extent that it is worthless for its intended purpose. The result may be a porous joint. In order to forestall the ready argument that "you would never get a dam built if you prohibited concrete placement in the rain," it may be noted that rolled earth dams are often built in damp climates and the nature of equipment used in their construction definitely limits the work to dry weather. Admitting the validity of the argument quoted does not alter the fact that serious damage may and sometimes does result from the common practice of paying too little attention to the effects of rain on the placement of concrete.

Cleanup, Curing, and Finishing.—Preparation of concrete surfaces upon which additional concrete is to be placed is considered to be a part of the general problem of "Construction Joints," which subject is presented by Mr. Steele in this Symposium. It is sufficient to note that construction joints are probably the weakest locations in any concrete dam, and consequently their treatment deserves very careful consideration.

The effects of concrete quality and placement methods on effectiveness of construction joints are fundamental. In addition to causing surface laitance accumulations, concrete that bleeds appreciably may seriously affect the upper layer of the concrete itself if the surface is disturbed. Agitation causes remixing of surface water with concrete near the surface and may result in a concrete of higher water ratio than that originally placed. Evidence of this has been noted in cores drilled across construction joints which, in many cases, break below, rather than at or above, the joint between lifts. A well-compacted surface with aggregate particles embedded in the concrete is essential to a good construction joint. In general, surface vibration should be avoided because of the danger of bringing mortar to the surface and remixing it with accumulated water. Traffic of placing crews and others over completed lifts should be prohibited until the concrete has hardened.

Curing of concrete includes its protection from extreme temperatures, drying out, and other possible damaging agents after the concrete has been placed in the forms. Protection must be afforded until the concrete has developed sufficient strength to withstand the effects of conditions under which it is to function. Protection from excessively high temperatures and drying out may usually be accomplished to best advantage by continuous sprinkling of surfaces. Forms left in place help greatly. With normal portland or modified cement the sprinkling should be continued for at least fourteen days. With low-heat cement, the time should be extended to at least three weeks and preferably longer. Protection from drying out alone may be afforded by coating concrete surfaces soon after removal of forms with certain sealing materials which retard moisture loss from the surfaces. Care should be exercised in selecting these materials, however, as only a few have been proved to be effective. Black coatings, when exposed to the sun, should be painted with some heat-reflecting paint to prevent excessive temperatures at concrete surfaces. Thick concrete sections are usually affected only slightly by drying out, except at the surface. Consequently, sprinkling of mass concrete surfaces has little to do with moisture-volume change or cracking inside the mass, except as surface cracks may furnish weaknesses and provide starting points for temperature cracking. Thin concrete sections are much more affected by surface-moisture loss because, in addition to affecting strength and durability of the surface, drying may remove enough moisture from the interior of the structure to produce large shrinkage and resulting cracking.

Cold-weather protection, often a problem with concrete containing normal or modified portland cement, presents considerably more difficulty when low-heat cement is used. The rate of hardening of any concrete is very slow at temperatures of 40° F or lower, and for placing temperatures approaching this range, a considerably longer time of protection against freezing must be provided than for placing temperatures 10° or 15° F higher. As it is undesirable to raise mass concrete temperatures above a minimum which is absolutely necessary (probably between 40° and 50° F), provision for protection against freezing for two weeks after placing may be necessary in some cases.

The essential requirements in obtaining a concrete finish surface that will withstand erosion and weathering are low water ratio and proper compaction. High cement content, undesirable from many standpoints, is usually necessary in order to provide the required quality of cement paste. In monolithic finishes on concrete lifts of appreciable depth, it is not possible to obtain a lasting surface if bleeding in the concrete takes place because of water gain in the top layer while working the surface with trowel or float. In all cases the minimum amount of work on the surface needed to produce the desired texture or appearance will also produce the most lasting surface. Concrete surfaces that can be made by placing concrete against forms are nearly always more superior in resistance to erosion and weathering than those finished by floating or troweling.

Cracking.—The dangers of cracking in a concrete dam may be very serious, affecting structural stability and the lasting qualities of the structure. Effects on stability are possibly less than might be expected because of tendencies to heal and reunite disconnected sections if openings are small and if water leakage or frequent movement does not interfere. Effects on durability are probably greater than might be expected because they are not immediately apparent. Cracks provide passages for percolating water and encourage the solution of cementing materials in the concrete. Presence of water in cracks permits the full disruptive forces of repeated freezing to accelerate the normal surface disintegration greatly.

Cracking in concrete may be due to temperature effects, moisture volume changes, foundation deflections, load deformations, or a combination of these. In thick concrete dams it seems conservative to estimate that more than 75% of all cracking is due to temperature effects introduced during the construction period. In multiple-arch and other hollow dams the effect of moisture volume changes in thin arch sections or slabs may be the largest factor contributing to cracking. Cracking in buttresses of moderate thickness in these types of dam may be caused by a combination of moisture and temperature changes. Cracking of concrete produced by foundation deflections or load deformations which are independent of temperature or moisture volume change effects present problems outside the scope of this paper.

Temperature stresses that produce cracking in thick concrete sections might be considered as being of two classes: Those produced by temperature changes in a block of concrete acting as an unrestrained unit, and those caused by restraint of one section by another. Advantages of attempting to distinguish between the two are of value for purposes of discussion only since basically all cracking is due to restraint of one kind or another.

Cracking due to temperature changes in an unrestrained block of concrete usually takes place early in its curing history, often within a few weeks after the block has been cast and sometimes within a few hours. In most cases such cracking takes place while the interior of the block is still rising in temperature, although sudden cold weather may produce the same effect at much later ages when internal temperatures are falling. It is due to inability of the cooling layers near exposed faces to elongate with the inner layers which may be increasing in volume due to temperature rise. Tendency to produce such cracking is governed by properties of the concrete, changes in temperature, and the time through which temperature changes take place. Insulating effect of forms, effect of curing water, and concrete placing temperatures are partly controllable factors that may influence such cracking. The most serious uncontrollable factor is varying air temperature. It has been observed that concrete placed early in the spring season of the year is much less subject to cracking than that placed in summer or early fall because of generally favorable temperature conditions during seasons of rising temperature and unfavorable conditions during seasons of falling temperature.

Cracking due to restraint such as may be produced in concrete placed on rock foundations or in concrete that has cooled to nearly mean annual temperature is caused by tensile stresses set up during cooling of concrete above the joint. From a structural point of view, cracks produced in this manner are more serious than those previously discussed because they are usually vertical and parallel to the axis of the dam, approximately in planes of maximum shear if they occur near the toe of the dam. Little is known about the development of these cracks because there is seldom opportunity for observation of their behavior except in hollow-dam buttresses or concrete walls where moisture volume changes may also be influential. It is likely that cracks started initially by temperature changes between interior and exterior are continued by a restraining action of the foundation during cooling of the concrete mass, resulting in cracks of zero opening at foundation and appreciable opening some distance above. In some instances such cracks have been known to extend upward to the downstream face of the dam.

Cracking of concrete in thin sections due to moisture volume changes is difficult, if not impossible, to eliminate entirely, but may be minimized by careful design of the structure and excellence of concrete quality. Proper spacing of artificial contraction joints, where structural requirements permit, will aid materially in controlling such cracking. Using highly impermeable concrete of high tensile strength will also aid greatly by slowing the rate of moisture change and by providing concrete more resistant to whatever tensile stresses are produced. Protection against moisture loss from concrete during early ages, by applying moisture-proofing membranes to exposed surfaces, has been at least partly successful; but so far as is known, little has been accomplished toward extending their effectiveness over long periods. Moist curing or its equivalent is essential during early ages.

Properties of materials which affect temperature cracking in concrete include the following: Rate of heat liberation and total amount of heat released by the cement during hardening, coefficient of thermal expansion of concrete, elastic modulus of concrete, tensile strength of concrete, plastic flow of concrete, and thermal properties of concrete. These items, together with foundation characteristics, construction scheduling and procedures, and weather variations, govern the behavior of a concrete structure in regard to temperature cracking. Efforts to eliminate the cracking by taking advantage of controllable factors affecting temperature changes have not been entirely successful to date, but results of such efforts have been most encouraging. Of the procedures most effective in producing conditions favorable to decreased temperature cracking, the following are considered to be of particular importance:

1. Use of low-heat cement or its equivalent in the lowest amount consistent with strength and other requirements (this usually means providing one concrete mix for the interior and another for exposed faces of the structure);
2. Use of low concrete-placing temperature by cooling mixing water or other ingredients when the placing temperature exceeds perhaps 50° F;
3. Elimination of extended exposures of long and high contraction joint faces;
4. Providing especial consideration to irregular foundations;
5. Limiting concrete lifts to a 5-ft thickness, or less, and reducing this appreciably when covering foundation or concrete which has been allowed to cool;

6. Scheduling of concrete placing so that placing rates are uniform, with sufficient time between lifts to permit dissipating an appreciable amount of the heat liberated by hardening of the cement; and

7. Artificially cooling the concrete in place.

For extremely large dams, artificial cooling is almost necessary if groutable contraction joint openings are to be obtained within a reasonable length of time after completion. Therefore, control of cracking may be accomplished best by taking full advantage of the artificial cooling, with as many of the other precautions as may be considered necessary. The effectiveness of artificial cooling on any concrete of mass proportions is well established, but to take full advantage of its effects the process should be started as soon after the placing of concrete as conditions permit—within a day or two if possible. For small dams, or dams of moderate size, a proper combination of the foregoing procedures can be made to reduce temperature cracking to a small amount, if not to eliminate it entirely, even without the use of artificial cooling.

RECORD TESTS

Attempts to investigate old concrete structures on which no records of construction methods or materials tests are available furnish convincing evidence that future improvement in dam construction will be due in no small way to information gained from test data and construction records kept on present projects. Advanced practice in dam building is to investigate materials and concrete thoroughly, well in advance of construction, and to continue with record testing as the dam is built, thus leaving valuable information for future study in connection with the behavior of the structure. Preliminary investigations are often of an involved nature, including tests not well adapted to continuation as routine record testing. Compressive strength is still considered to be the best single index to concrete quality, provided relations between compressive strength and other qualities of the concrete exist and may be reasonably well established, even if the actual meaning of a compression test remains obscure. It is usual, therefore, to make compressive strength one of the more important record tests on concrete and to establish relations between strength and other properties whenever possible. Other common record tests include acceptance or equivalent tests on cement and aggregates and such investigations as may be found necessary by unforeseen effects of plant or methods on quality of concrete. In preliminary studies the various properties of concrete that may affect the behavior and lasting qualities of the completed structure are included. Usually considered, if not investigated, are properties as follows: Durability; workability and bleeding of fresh concrete; cement requirement; unit weight; compressive strength; elastic and plastic properties, Poisson's ratio; permeability; volume change due to temperature and moisture changes; thermal properties; and temperature conditions in masses.

Undoubtedly, increased knowledge of the properties of concrete as determined by laboratory tests has been responsible for greatly improving the quality of dams in general, but recent investigations on dams during construction and subsequent loading have shown that the complete story of the structural properties of concrete can be learned only from studies of the full-size structure. Such investigations may well be considered a part of the record testing for an individual dam because they show whether or not the material behaves according to the intentions of the designers. They also serve the even more important purpose of furnishing general information on what actually takes place inside of a dam subjected to temperature changes and water load. Data from such investigations cover temperature, strain, deflection, joint openings, cracking, uplift, leakage, and other phases of dam performance. A considerable amount of such work has been in progress for some time and interesting and significant results have been obtained. Future investigations, particularly in the direction of stress-strain analysis and studies of temperature cracking in mass concrete, may be expected. It is at least an interesting fact that a 265-ft gravity dam has been known to deflect upstream ⅜ in., against a rise in lake level of 150 ft, due to seasonal temperature effects.

Conclusion

The art of building concrete dams is becoming a science. Realization by engineers that concrete is a material that changes with variable quantities of its ingredients, and with manufacturing, handling, placing, and curing procedures, has done much toward attaining this objective. Efforts toward gaining control of the variables known to exist in concreting materials and procedures, and of first importance in producing structures of uniformly high quality, are becoming more successful, particularly on the larger dams. Investigations leading to a better understanding of the behavior of concrete under conditions of weathering, water pressure, loads, temperature and moisture changes, and other variables, are furnishing designers of dams with much needed information for future progress. There is still obvious need for further improvement in practically all phases of dam design and dam construction, but advances made in the past few years indicate that future progress may be even more rapid.

APPENDIX

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(11) **"Drying Shrinkage of Concrete as Affected by Many Factors,"** by Roy W. Carlson, A. S. T. M., 1938.

CONSTRUCTION JOINTS

BY BYRAM W. STEELE,[37] ASSOC. M. AM. SOC. C. E.

SYNOPSIS

The construction joint in concrete structures has taken on a new and added importance in recent years because of the tendency toward increased cracking caused by modern speed of construction and other factors. There is also gradually developing in the construction world a desire to substitute for the ragged haphazard crack, which is so objectionable from the standpoint of safety and appearance, a designed crack which will be architecturally acceptable and which will not develop undesirable features at a later date. Thus it appears that the construction joint, in its every phase, even if it is a necessary evil, should receive more careful consideration in the future than in the past. Where labor is cheap and materials are costly, this matter will undoubtedly receive more consideration than where labor is high-priced and materials relatively inexpensive.

A construction joint as treated in this paper is a formed or unformed, horizontal, vertical or inclined surface between masses of concrete placed at different times. The spacing, form, and method of construction of such joints are the leading topics considered herein.

On the question of the proper spacing of construction joints in general, and of vertical transverse joints in arch and gravity dams in particular, there is universal concurrence that they have been placed too far apart in the past and should be spaced closer together in the future unless other means are used to guarantee the elimination of cracking. It is also generally conceded that the spacing of joints for each dam should be decided upon only after due consideration of the type and height of the dam, profile of the dam site, foundation materials and conditions, speed of construction, climate, architectural treatment, whether artificial cooling of the concrete or aggregates is to be resorted to, the heat flow characteristics of the aggregates themselves, water-cement ratio of the concrete, height of the lifts, time interval elapsing between lifts, and last, and most important, the type of cement. There does not appear, however, in all the available literature on this subject, a set of rules for the spacing of joints that seems to be acceptable for general application.

It is interesting to note that 2,000 years ago the Romans were having much the same trouble with hydraulic structures that modern engineers are experiencing today and that the human tendencies then were just about on the same level

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as now. In A. D. 97, Sextus Julius Frontinus, water commissioner of the City of Rome, wrote as follows:[38]

> "The heat of the sun is no less destructive to masonry than is too violent frost. Nor is greater care required upon any works than upon such as are to withstand the action of water; for this reason, all parts of the work need be done exactly according to the rules of the art, which all the workmen know, but few observe."

The data upon which the conclusions of this paper are based are assembled for reference in Table 11 and in the supporting comments in the Appendix.

JOINT SPACING

It seems to be quite generally recognized that in dividing a dam into monoliths or blocks, the ideal spacing of the vertical construction joints normal to the axis of the dam would be a dimension that is varied from block to block to suit the profile of the dam site and the type and height of the dam. This ideal spacing is possible, but, due to serious objections from the architectural and construction standpoints, is rarely ever adopted. Economic considerations dictate that forms must be paneled and re-used until worn out, thus limiting the use of an ideal spacing, especially if variable.

It is believed, however, that a more rational approach to this subject than is generally made could be accomplished in connection with the construction of any dam if a construction-joint-spacing study was conducted early in the design period, before such features as outlet works, spillway gates, and a bridge over the dam begin to take form. Thus it would be possible to have sufficient data on hand to permit adequate consideration of the spacing of joints at the same time that bridge spans and spillway and outlet features are being considered. Regardless of the latter, however, a dam should be made structurally sound before being made architecturally beautiful.

A few years ago, 150 ft was not an unheard of spacing for construction joints in dams, and 75-ft to 100-ft spacing was common practice. At present, 15 m (49¼ ft) in foreign countries and 50 ft in the United States seem to be the most popular spacings for the vertical joints normal to the axis of the dam, with the general requirement that each joint shall extend entirely through the structure. A notable exception to this generalization is the case of the Cignana Dam[39] in Italy, completed in 1928, in which there are three distinct systems of joints, as shown in Fig. 9. The principal ones are 30 m (98.43 ft) apart, cutting the entire section of the dam; the secondary ones intersect the entire width of the dam but extend only 19 m (62.34 ft) below the crest; and finally, a partial secondary system of joints midway between the principal and secondary extend only 19 m below the crest and from the upstream face to a drainage well, which is a short distance from the upstream face. Cignana Dam was the object of a series of observations to demonstrate the physical and mechanical behavior of the structure during and after the setting of the concrete, especially the joint openings for the different types of joints and at different seasons of the year, and in this respect is one of the most interesting demonstrations of record.

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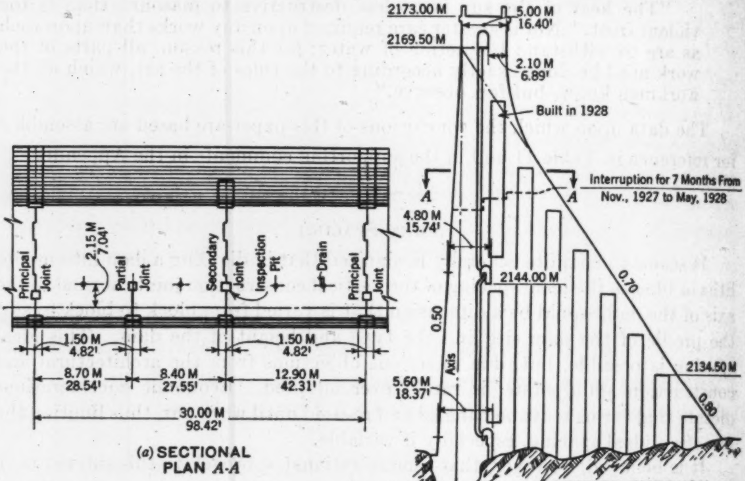
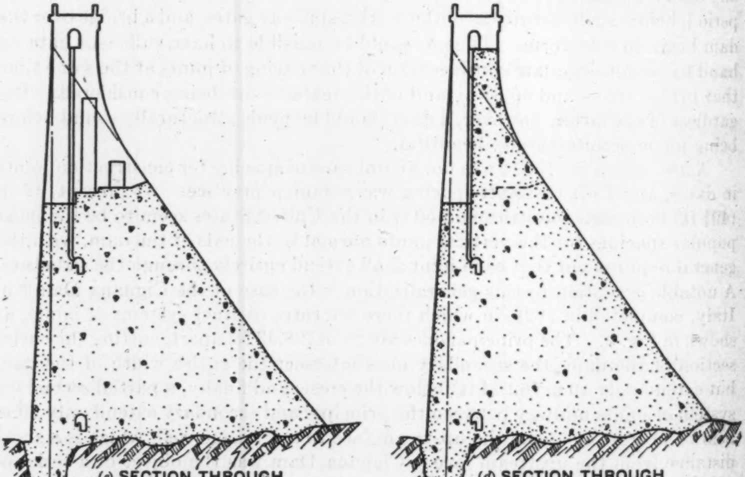
[38] Excerpt from "The Water Supply of the City of Rome," by Sextus Julius Frontinus.

[39] "Longitudinal Contractions and Expansions Measured in a Large Concrete Dam," by Felice Contessini, *Transactions*, 2d Cong. on Large Dams, International Commission on Large Dams of the World Power Conference, Supt. of Documents, Washington, D. C., 1938, Vol. III, p. 161.


2173.00 M
5.00 M
16.40'
2169.50 M
2.10 M
6.89'
Built in 1928
A
A
Interruption for 7 Months From
Nov., 1927 to May, 1928
4.80 M
15.74'
2144.00 M
0.70
0.50
Axis
2134.50 M
0.80
5.60 M
18.37'
Principal Joint
2.15 M
7.04'
Partial Joint
Secondary Joint
Inspection Pit
Drain
Principal Joint
1.50 M
4.82'
1.50 M
4.82'
8.70 M
28.54'
8.40 M
27.55'
12.90 M
42.31'
30.00 M
98.42'

(a) SECTIONAL PLAN A-A

(b) SECTION THROUGH PRINCIPAL JOINT



(c) SECTION THROUGH SECONDARY JOINT

(d) SECTION THROUGH PARTIAL JOINT

FIG. 9.—CONSTRUCTION-JOINT LAYOUT—CIGNANA DAM

There are numerous examples in dams, both in the United States and in other countries, of the lack of adequate consideration of the location of the vertical joints between monoliths on the canyon walls. It is common knowledge that cracks invariably occur normal to the canyon wall slope rather than vertical, and yet engineers insist on the widest spacing that can be justified, and on carrying the vertical joints to rock in the most unnatural locations, if cracking is to be avoided—all for the sake of uniformity! True, it would not be practicable to form inclined joints so as to be normal to the canyon walls, but it is possible to locate and space vertical joints so that the tendency for inclined cracks to form is a minimum.

Perhaps this is a good place to admit that large monoliths can be built without objectionable cracking, but there are still so many "ifs" in the road to stumble over that to advocate returning to wide-joint spacing does not seem to be a desirable attitude to assume except for the dam where the necessary niceties of concrete control will be practiced without fail.

In connection with joint spacing, volume change, and the multitude of details that attend the raising of dams such as Assuan Dam, in Egypt, and O'Shaughnessy Dam, in California, there has been some very interesting and comprehensive investigation into the realm of actual construction joint performance.

After making a comprehensive survey of cracks in twenty-one dams in the western part of the United States, H. M. Westergaard,[40] M. Am. Soc. C. E., states that "By observing cracks and joints in a number of dams one is impressed with a certain regularity of the phenomena" but adds that "No single distance can be stated defining the spacing of the vertical expansion joints necessary to avoid major cracks between them. As a general rule, a smaller height of the dam requires a closer spacing."

The trend toward closer spacing of joints is not confined to concrete dams; it is also evident in concrete pavement construction. E. F. Kelley, chief, Division of Tests, U. S. Bureau of Public Roads, concludes[41] that "Reasonable assurance of the absence of the transverse cracking in concrete pavements can be obtained only by the use of slabs that are about square with dimensions of the order of 10 feet."

In regard to the spacing of longitudinal joints in a dam, the same principles apply that have been discussed for transverse joints. From the standpoint of structural sufficiency, longitudinal cracks in a dam are far more objectionable than transverse ones, since the latter principally affect the watertightness qualities of the dam, whereas the former may divide the dam into parts that would not be stable, especially if hydrostatic pressure builds up to reservoir head in the crack. Longitudinal joints have been used in the construction of a few dams such as Assuan, Boulder, and Shasta, but the intricacies forced into the construction procedure because of the necessity for grouting longitudinal joints have made this type of joint relatively unpopular and one that should be avoided, if practicable.

In a straight gravity dam the opening of longitudinal joints, or the formation of irregular longitudinal cracks where there are no longitudinal joints, may or may not be a serious problem, but it is certain that such openings should be avoided, if possible, in the interest of the permanent safety and integrity of the structure. Where at all practicable, it would appear preferable, even at considerable additional expense, to construct a straight gravity dam without longitudinal joints. The production of an ideal uncracked monolith is possible if the temperature of the material of which it is constructed is controlled so as to avoid initial high temperatures and high temperature differentials in short distances. The latter condition starts cracks at the surface, which, once started, are as difficult to stop as the crack in a windowpane.

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[40] "Cracks Observed in Dams," by H. M. Westergaard, *Technical Memorandum* of Bureau of Reclamation, July 8, 1930.

[41] "Application of the Results of Research to the Structural Design of Concrete Pavements," by E. F. Kelley, *Journal*, Am. Concrete Inst., January, 1939, p. 437.

The construction procedure followed for crack elimination at Hiwassee Dam, in North Carolina, and the results produced are encouraging. Hiwassee is the first dam in which a combination of crack-elimination methods of construction has been followed consistently. Although some cracking at horizontal-lift joints was noted during the winter of 1940, the writer is advised that no vertical cracking on the faces of the dam has yet been experienced and that longitudinal cracking occurred in only a few monoliths.

Shallow lifts, proper type of cement, low-cement content, refrigeration of mixing water, sprinkling the coarse aggregate and blowing air through it to standardize moisture content and reduce temperature, restrictions on placing concrete during hot weather, sufficient time between lifts to permit a large amount of heat loss from the surface of the lift, and cooling with embedded pipes are the principal means of avoiding or minimizing the tendency to crack. All of these factors affect the cost of the finished concrete and hence must be weighed carefully as to cost and benefit before deciding upon specification limits for each factor.

Foundation conditions also may play a very important rôle in the formation of cracks. Each and every monolith in a straight gravity dam should be designed to stand as an independent unit on its own foundation. As the height of such dams increases, there appears a "no man's land" where there is an ever-present danger of longitudinal cracks parallel to the axis of the dam. This situation can be handled in one of three ways: (1) The monolith can be built so as to prevent the formation of temperatures within the mass that will form cracks during the cooling period; (2) the monolith can be built in columns with joints parallel to the axis of the dam that can be grouted after temperature stabilization; or (3) the monolith can be built with closing slots that can be filled with concrete after the surrounding mass has cooled naturally or has been cooled artificially. Obviously, the grouted longitudinal joint has some very definite structural weaknesses which expose it to possible future cracking and its attendant hazards; so has the slotted type of joint, but to a much lesser degree, and this weakness can be largely eliminated if reinforcement is extended into the slot from each side of the joint so as to make an interlocking reinforced mass of the closing plug and adjacent concrete. The proper form for this slot, the treatment of the surfaces of the concrete in the slot before the plug is poured, the details of the reinforcement, the season of the year and rate at which the plug is poured, the characteristics of the plug concrete as compared to the adjacent concrete, and many other minor points vitally affect the transfer of stress across an area which at best is not possessed of the same mass characteristics as the monoliths which it unites. Hence, it is firmly believed that the artificial control of temperatures as contemplated under item (1) is the most satisfactory, probably the only satisfactory, solution to a problem that confronts the builders of all high, straight gravity dams.

Foundations with faults or planes of weakness that might produce cracks parallel to the axis of the dam after the temperature has stabilized within the monolith are possible but not common and would require special design consideration which cannot be covered in a paper of this length.

PLAIN VERSUS KEYED JOINTS

Joints in dams are either plain or keyed and, if keyed, there may be few or many keys. The keys may be narrow or wide and shallow or deep, or a combination. Joints in which the keys are so dimensioned as to give a "fifty-fifty" theoretical shearing resistance are quite popular with many design agencies. The batter on the sides of the keys also is a point on which there is a wide range of preference, the batter varying from nearly perpendicular to the plane of the joint to as flat an angle as 30° off the plane of the joint. One has only to examine a few sets of drawings to be convinced that there is no particular "rhyme or reason" to keys and keyways, and that the designer's fancy has been given full expression in this respect.

As a deterrent to the passage of water, keys and keyways in the vertical transverse construction joints in dams may have some desirable features, since a keyed joint with slight batter on the sides of the keys will silt up more readily than a plain joint because the actual width of opening at the battered section is only a fraction of the width of the plain joint.

Obviously there is less likelihood of cracks starting with the plain joint than with the keyed joint since the reentrant angles in the latter are so often the starting points for cracks that develop due to surface differential temperatures during the construction period but later are the entering wedges for objectionable cracking parallel to the axis of the dam that might otherwise never develop.

It is believed that the keyed joint in concrete dams and retaining walls is a hang-over from the mortise and tenon so commonly used in timber construction. Theoretically, it may be relatively easy to justify the use of keyed rather than plain joints, but if a careful analysis is made of each case in question it is believed that the questionable features of many keyed joints would outweigh the unquestionable ones and that many keys now in common use would be omitted.

The question as to the desirability of keys in horizontal joints in concrete is an old and much discussed problem that has not yet been settled to the satisfaction of all, although it is now generally conceded that the elimination of keys on the horizontal lifts of mass concrete is desirable. With vibration, modern concrete control, and strict inspection, the leaky horizontal joints of the past have largely disappeared. It is now possible to secure good bond on nearly all horizontal surfaces, and hence the apparent need for some kind of an obstruction to the flow of water along the plane of the joint has disappeared. As for shear, it is believed that a rough, well-bonded surface is far superior for the transfer of stress than dependence on keys.

The proper kind of concrete immediately below the plane of the joint is still lacking in too many horizontal joints in thin walls. This is due primarily to working the surface layer of concrete too much, the result being a layer of concrete of high water-cement ratio and low density and strength. To attempt to work a horizontal key form block down into such a surface aggravates the matter still further and serves only to make bad matters worse. Hence it is believed that, if horizontal joints in thin walls were treated in the same manner and with the same care as horizontal joints in mass concrete dams, corresponding results would be obtained. If some form of stop or key is insisted upon for horizontal joints in thin walls, a ¼-in. steel plate 6 in. wide set across the joint will accomplish the desired result with the least disturbance to the finished concrete surface. It will then be much easier to secure satisfactory concrete below the joint, and to clean up the joint surface just before the next pour is started, than with a key. The less a concrete surface is disturbed after the concrete is placed and consolidated the better. In the final cleanup before concrete placement is resumed, wet sandblasting is the most effective method of producing a satisfactory surface on which to deposit concrete and secure good bond. Careful inspection will always be necessary on this class of joints to prevent rock pockets at the surface of contact and in the corners. A half-inch layer of mortar of the same properties as exist in the concrete will be helpful in securing good horizontal joints. This mortar, however, should be deposited immediately ahead of the batch of concrete so that it will not stiffen up and prevent the coarse aggregate particles from bedding satisfactorily.

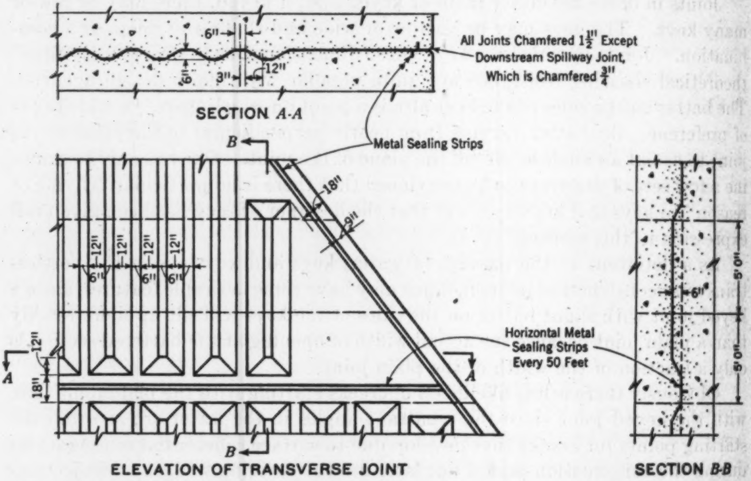
6''
6''
3''
12''
All Joints Chamfered 1½'' Except
Downstream Spillway Joint,
Which is Chamfered ¾''
SECTION A-A
Metal Sealing Strips
B
18''
9''
12''
12''
12''
12''
6''
6''
6''
12''
18''
A
A
B
Horizontal Metal
Sealing Strips
Every 50 Feet
5' 0''
5' 0''
6''
ELEVATION OF TRANSVERSE JOINT
SECTION B-B

FIG. 10.—TYPICAL CONSTRUCTION-JOINT DETAILS FOR DAMS OF THE BUREAU OF RECLAMATION

Dams such as Boulder, Grand Coulee, and Shasta bring into the construction joint feature many problems aside from strictly technical ones. In the construction of Boulder Dam, the details shown in Fig. 10 were developed for the radial joints. They proved very satisfactory and have likewise been used in the construction of Grand Coulee, Shasta, and other dams. Although the forming of keys is only a detail of form work, it is one of the details that affects the cost quite materially if it is not standardized to the point where rapid shifting of forms is permitted, and re-use with long life is guaranteed. The details shown in Fig. 10 may appear rather complicated, but this key system has proved to be more satisfactory than any other system with which the writer is familiar. Similar form details[42] were also used in the construction of Seminoe Dam, in Wyoming.

SLOTS VERSUS GROUTED JOINTS VERSUS OPEN JOINTS

An ideal dam should have neither transverse nor longitudinal joints or cracks. Whether this ideal is possible of attainment is open to question due to the fact that volume change in the concrete of the dam and volume change in the foundation supporting the dam are possibilities that are always present.

Whether the volume change contemplated in the preceding paragraph and the large variety of cracks it produces are so objectionable as to warrant expensive construction procedure to avoid them is also open to question. However, there is no question but that the ideal crackless dam (especially as regards longitudinal cracks) is preferable and is worth considerable study to attain in so far as practicable.

In recent years, since volume change in concrete has been better understood, there has been a concerted effort by many agencies to render both arch and gravity dams more nearly monolithic than has been their condition in the past, in an attempt to obtain better distribution of stress and closer agreement between analytical assumptions and actual conditions. This has been accomplished through the use of closing plugs between monoliths, or the grouting of the joints between the monoliths after the latter have cooled to approximately minimum annual temperature. Closing plugs, or slots, are ordinarily 3 ft thick and filled with concrete similar to the concrete in the adjacent monoliths.

The necessity for grouting the transverse joints in a straight gravity dam appears questionable. Instead, it would appear that each monolith should be stable on its own base. To attempt to modify the distribution of stress in a straight gravity dam by grouting the transverse joints appears to have too many questionable features to warrant the practice.

In a curved gravity dam, and in arch dams, conditions may be somewhat different from those in a straight dam, and it may be economically desirable to take advantage of the transfer of stress from gravity to arch elements. In that event, the grouting of joints or the use of closing slots is vital to the theoretical accomplishment of the stress transfer if, for instance, tension cracks near the base of the dam on the upstream side are to be avoided. Either method will yield the desired results if satisfactory construction and grouting details are developed. There are advantages in both systems and hence both systems should be given due consideration. The slotted method of treatment seems to be favored in foreign countries, whereas joint grouting is the system preferred in the United States. Flood diversion requirements during construction may render the use of slots with arch dams impracticable because of the possibility of sudden filling of the reservoir and the collapse of the unsupported arch monoliths.

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[42] *Western Construction News*, March, 1939, p. 98.

In commenting on this subject, A. Thimel (director general of the Department of Public Works, in France) concludes[43] that for arch dams "a system of slots with keys is best to ensure the bonding of the blocks," and that for curved gravity dams "The system of radial slots with keys is preferred * * * and shrinkage joints become unnecessary." He also draws the conclusions from experience to date "that only a few elementary and now well-defined precautions are required when designing joints of a large dam" and that "if these precautions are taken, the joints will never be one of the weak points of the dam." The latter statement is accepted heartily; and in so doing the writer has in mind one dam in the United States which is an excellent example of satisfactory and watertight concrete, but the joints were spaced so far apart that the objectionable cracking and leakage have overshadowed the other good qualities entirely.

In describing the Mareges arch dam, A. Coyne (chief engineer of bridges and highways, in France) states[44] that "the dam was divided into blocks separated by slots one meter wide, and that to insure watertightness copper stops were installed at the upstream face and a system of horizontal and vertical grout pipes installed, and the joints between the slot and the adjacent blocks grouted." In the United States at Boulder Dam under similar circumstances the 8-ft cooling slot in the middle of Boulder Dam was filled with concrete, after the dam had been cooled to its mean annual future temperature, without grouting the joints on either side and with no apparent leakage or other undesirable effects.

At the time the contract was let by the Bureau of Reclamation for the construction of the Gibson Dam, in Montana, the plans provided for closing slots or plugs the full height of the dam between certain pairs of monoliths, these to be later filled with concrete. Subsequently, this plan was abandoned and the joints grouted by means of the pipe system which has since been adopted as standard practice by that Bureau. Since the joints in Gibson Dam were grouted, Deadwood, Owyhee, Cat Creek, Boulder, Parker, Madden, Grand Coulee, and Seminoe dams have been treated in much the same way. With the joint surface clean and suitable for the adherence of a grout film, there does not appear to be any valid reason why the radial grouted joint in the interior portions of the dam should not be permanently satisfactory, provided the grout film is introduced under sufficient pressure to drive the excess water into the pores of the adjacent mass concrete so as to create a dense, durable, and impermeable film. Grout films not produced under pressure are likely to be chalky and to disintegrate easily.

Thin arches, which will cool to mean annual temperature throughout the entire mass during the construction period, would seem to offer the most attractive field for the slot rather than the grouting method of joint treatment, since the constant change in surface and near-surface temperatures may, in time, affect the grout film adversely by rupturing its bond with the adjacent concrete, thus contributing to its ultimate disintegration and the consequent increase in arch stress due to decreased arch thickness.

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[43] "Joints de Retrait et Joints de Contraction et Dilatation," by A. Thimel, *Transactions*, 2d Cong. on Large Dams, International Commission on Large Dams of the World Power Conference, Supt. of Documents, Washington, D. C., 1938, Vol. III, p. 61.

[44] "Clavage des Barrages Voûtes," by A. Coyne, *loc. cit.*, p. 91.

Decreased arch thickness, as noted in the preceding paragraph, and the decrease in arch thickness resulting from the installation of grout stops, are not serious matters for arches of appreciable thickness since the worst that will probably happen is that internal arch stresses will be increased and the distribution changed somewhat, all of which can be taken care of by the normal or usual factor of safety. In thin arches, or in the upper part of arch dams in which this decrease is an appreciable part of the total arch thickness, provision should be made for increasing the total thickness so as to provide for satisfactory stresses in the part of the arch joint that actually remains in contact under maximum load conditions.

When flood diversion during construction is adequate, and where time for the cooling of the blocks or monoliths of the arch permits, it is believed that due consideration should be given to the use of slots or closing plugs instead of grouted joints. Under this method of construction, there will result a mass of concrete that is more nearly permanently monolithic than under the joint grouting system where constant change in volume near the surfaces of the dam may ultimately break down the bond between the grout film and the adjacent concrete and thus gradually result in the disintegration of the film. This may or may not be of any very great moment, but, if the film is not as permanent as the adjoining concrete, it is not worth putting in the joint in the beginning. The interior of the dam, beyond the effect of annual temperature cycles, however, is not subject to the foregoing criticism and, if satisfactorily grouted, the film should be as permanent as the adjoining concrete.

However, for the surface layer of concrete in any type of dam in which there is a daily and annual temperature fluctuation, the permanency of a thin film of grout in the joint is open to serious question, and the possible ultimate disintegration of this film would lead one to conclude that the slot method of joint construction might be more permanent and satisfactory than the grout-film method.

JOINT SURFACE TREATMENT

Where future differential movement of the surfaces of a vertical joint parallel to the plane of the joint is anticipated, the use of some form of asphaltic or bituminous compound with or without asbestos fiber may be desirable; but even here the indiscriminate use of asphaltic or bituminous paints and putties on joint surfaces is not founded on necessity. If the keys in a joint are properly shaped or tapered so as not to "hang up" when the adjacent surfaces of a block separate, due to contraction, there is little need for any kind of surface treatment. However, if movement is anticipated between joint surfaces parallel to each other, the keys and keyways near the outside surface of the structure may give trouble by causing concentration of stress that will produce cracks. This type of possible movement calls for adequate study and in certain cases for provision for surface treatment at critical points and possibly for a compressible filler where keys might shear out slabs of the surface concrete.

Bonding Joint Surfaces

The adequate bonding of horizontal and inclined concrete joint surfaces has been one of the most difficult problems to solve satisfactorily that has been encountered in the concrete manufacturing process. Bonding vertical surfaces is not as common a requirement as for inclined and horizontal surfaces, but the same principles apply and, where necessary, are more easily applied.

During recent years horizontal joint cleanup with a high-velocity, air-water jet, applied during the period between initial and final set, has largely displaced other methods of joint surface treatment, and when applied at the proper time and with the correct technique has produced excellent results.

Before the introduction of vibration, the roughing-up of joint surfaces (covering them with a coat of grout and then depositing thereon concrete of a little higher slump than later used for the overlying mass) was the accepted practice. Since the development of the vibrator and the acceptance by the construction industry of the practicability of placing 1-in. to 1½-in. slump concrete, many of the objectionable features of horizontal concrete joints have disappeared. Cores drilled across these joints reveal a more satisfactory density and a greater uniformity of aggregate distribution than formerly, without so much of the old-time tendency toward concentration of a matrix with too high a water content immediately below the lift joint.

Metal strips at horizontal construction joints, raised or depressed keys, and elaborate cleanup requirements are gradually disappearing from construction practice. However, in Claytor Dam in Virginia, completed in 1939, the horizontal construction joints were provided with a raised key (24 in. by 6 in.), a copper-bearing steel plate water stop (⅛ in. by 10 in.), and a formed open drain (8 in. deep by 10 in. wide), the center lines of which three items are distant 3 ft, 5½ ft, and 7½ ft, respectively, from the upstream face of the dam. The longitudinal drains stop short of the transverse contraction joints between blocks and are connected to the interior drainage system of the dam. This design of the joint itself is accompanied by a joint-treatment specification requiring the entire surface of each horizontal lift to be "cut back" a minimum depth of 1 in., and, for a strip 18 in. wide around all permanently exposed faces and the upstream face of the dam, the "cutback" must vary between 2 in. and 5 in. in depth, depending on the slump and the height of lift. The "cutback" normally was accomplished prior to initial set, with air or water jet, or both, under pressure, or by shoveling. No "cutting back" was permitted between the time of initial and final set, "cutting back" after final set being accomplished to the prescribed depth by the use of hand-picks or equivalent means. Thus it is apparent that the telltale examples of past poor construction practice have not yet been forgotten by some agencies and that this phase of construction procedure is still of major concern if satisfactory long-time performance is earnestly desired.

Observations over a long period of years, supported by recent tests by the Bureau of Reclamation, indicate that the less freshly placed concrete is disturbed after consolidation and prior to hardening, the better the qualities of bond and watertightness will be, and that the only cleanup necessary for a properly designed mix, properly placed, is that done just prior to the placing of another layer of concrete. After a thorough study of this phase of concrete placement, R. F. Blanks (engineer, Bureau of Reclamation) concludes[45] that the most desirable results with horizontal joints will be obtained if the following general rules are adopted:

(1) Do not permit placing concrete which bleeds and forms laitance excessively.

(2) Avoid excessive working of fresh concrete at the construction planes.

(3) After the concrete is once placed, do not permit it to be disturbed until it has attained sufficient strength to withstand traffic—omit the initial cleanup and keyways.

(4) Keep the surface continuously moist until the next layer is placed or until sufficient hydration of the cement has been obtained (where practicable, wet sand as a curing medium is preferable to any other).

(5) Clean the old surface thoroughly just prior to placing the next lift. The most satisfactory method is by wet sandblasting and washing. If the surface is essentially free of laitance and coatings, good results can be obtained by thorough wire brushing and washing. Where practically ideal placing has been obtained and the time interval before placing the next lift is not more than three days, high-velocity water jets may be sufficient. Chipping should be used only in extreme cases where damaged or defective areas are encountered.

(6) Thoroughly and effectively, broom into the old surface a layer of suitable mortar and place new concrete immediately.

The *Concrete Manual* of the Bureau of Reclamation illustrates[46] the condition of placement at a horizontal joint that is responsible for a great deal of the lack of watertightness in concrete and the characteristic surface disintegration found at and immediately below the horizontal lift joints in so many hydraulic structures.

In 1939, Charles E. Wuerpel[47] (engineer, Central Concrete Laboratory of the North Atlantic Division, U. S. Engineer Department, U. S. Military Academy, West Point, N. Y.) discussed and illustrated the results of a series of freezing and thawing tests of sixty-seven cores drilled across horizontal construction joints. It is interesting to note that, in some of these cores, although the joint itself remained sound, the deterioration immediately below the joint was much greater than in the remaining portion of the core. This indicates that, although the horizontal construction-joint cleanup and bond are being taken care of satisfactorily in the more recent hydraulic structures, there is still too much of a tendency to let an inferior grade of concrete accumulate in the upper portion of a concrete lift. For this reason it is believed that Mr. Blanks' rule (2) cannot be overemphasized and that some additional research along the line of Mr. Wuerpel's experiments would reveal some very definite trends that are worth noting for future durability.

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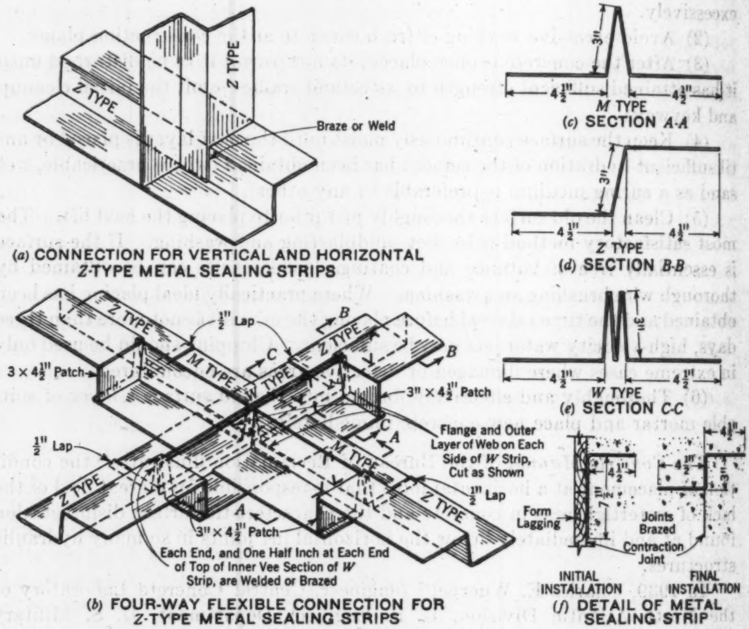
[45] "Treatment of Horizontal Construction Joints in Concrete to Improve Bond, Shearing Resistance and Watertightness," by R. F. Blanks, *Memoranda* to the Chf. Designing Engr., Bureau of Reclamation, March 8 and May 28, 1938 (not published).

[46] *Concrete Manual*, Bureau of Reclamation, U. S. Dept. of the Interior, March, 1939, Fig. 68, opposite p. 210.

[47] "Tests of the Potential Durability of Horizontal Construction Joints," by Charles E. Wuerpel, *Journal*, Am. Concrete Inst., January, 1939, p. 181.

WATER STOPS

An absolutely watertight permanent membrane of appreciable thickness would be desirable on the upstream face of any dam, but whether it is necessary and worth the cost is open to question.


Z TYPE
Z TYPE
Braze or Weld
3"
$4\frac{1}{2}$"
$4\frac{1}{2}$"
M TYPE
3"
$4\frac{1}{2}$"
$4\frac{1}{2}$"
Z TYPE
Z TYPE
$\frac{1}{2}$" Lap
B
Z TYPE
C
M TYPE
W TYPE
B
$3 \times 4\frac{1}{2}$" Patch
3" × $4\frac{1}{2}$" Patch
3"
$4\frac{1}{2}$"
$4\frac{1}{2}$"
W TYPE
C
A
$\frac{1}{2}$" Lap
Flange and Outer Layer of Web on Each Side of W Strip, Cut as Shown
Z TYPE
M TYPE
Z TYPE
$\frac{1}{2}$" Lap
A
3" × $4\frac{1}{2}$" Patch
Each End, and One Half Inch at Each End of Top of Inner Vee Section of W Strip, are Welded or Brazed
$4\frac{1}{2}$"
$4\frac{1}{2}$"
$4\frac{1}{2}$"
Form Lagging
Joints Brazed
Contraction Joint
INITIAL INSTALLATION
FINAL INSTALLATION

(a) CONNECTION FOR VERTICAL AND HORIZONTAL Z-TYPE METAL SEALING STRIPS

(b) FOUR-WAY FLEXIBLE CONNECTION FOR Z-TYPE METAL SEALING STRIPS

(c) SECTION A-A

(d) SECTION B-B

(e) SECTION C-C

(f) DETAIL OF METAL SEALING STRIP

FIG. 11.—METAL SEAL DETAILS, BUREAU OF RECLAMATION

A. Renaud (chief engineer of roads in France) summarizes French experience in a paper on joints in gravity dams by stating,[48] "A close examination, contrary to what would appear at first sight, shows that, in general, joints are sufficiently water-tight." E. H. Link (director of public works in Germany), in commenting on German experience in the spacing of joints in straight gravity dams, states:[49] "Double sealing is recommended for tightening the joints; single sealing has not always been sufficient."

In the original outline of this paper, it was planned to prepare schematic and enlarged detail plans to illustrate typical water-stop installations in dams in various countries, so that their relative dimensions and peculiarities could be

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[48] "Les Joints des Barrages-Gravité," by A. Renaud, *Transactions*, 2d Cong. on Large Dams, International Commission on Large Dams of the World Power Conference, Supt. of Documents, Washington, D. C., 1938, Vol. III, p. 19.

[49] "Entstehung und Abdichtung von Schwind-, Zusammenziehungs- und Dehnungsfugen in Staumauern," by E. H. Link, *loc. cit.*, p. 109.

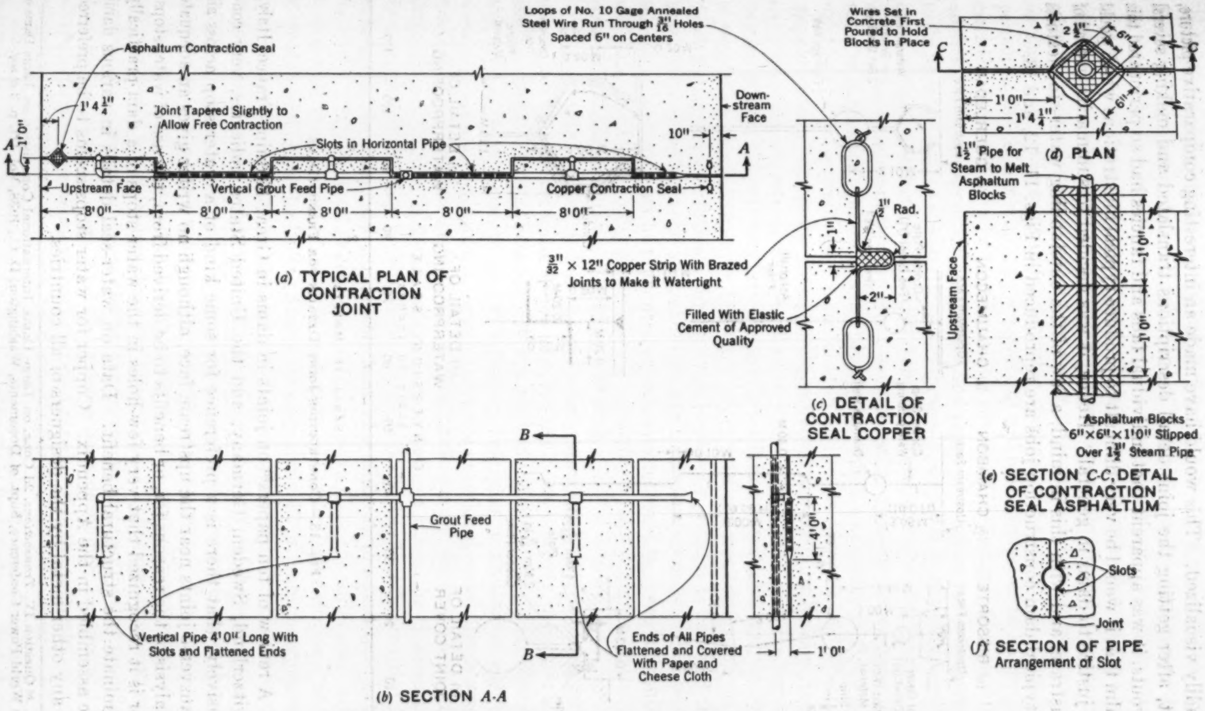
Asphaltum Contraction Seal
Joint Tapered Slightly to Allow Free Contraction
Slots in Horizontal Pipe
Down-stream Face
A
A
1' 0"
1' 4 1/4"
10"
Upstream Face
Vertical Grout Feed Pipe
Copper Contraction Seal
8' 0"
8' 0"
8' 0"
8' 0"
8' 0"
(a) TYPICAL PLAN OF CONTRACTION JOINT
Loops of No. 10 Gage Annealed Steel Wire Run Through 3/16" Holes Spaced 6" on Centers
1/2" Rad.
1"
2"
3/32" × 12" Copper Strip With Brazed Joints to Make it Watertight
Filled With Elastic Cement of Approved Quality
(c) DETAIL OF CONTRACTION SEAL COPPER
Wires Set in Concrete First Poured to Hold Blocks in Place
C
C
2 1/2"
6"
6"
1' 0"
1' 4 1/4"
(d) PLAN
1 1/2" Pipe for Steam to Melt Asphaltum Blocks
Upstream Face
1' 0"
1' 0"
Asphaltum Blocks 6"×6"×1'0" Slipped Over 1 1/2" Steam Pipe
(e) SECTION C-C, DETAIL OF CONTRACTION SEAL ASPHALTUM
Slots
Joint
(f) SECTION OF PIPE
Arrangement of Slot
B
B
Grout Feed Pipe
4' 0"
1' 0"
Vertical Pipe 4' 0" Long With Slots and Flattened Ends
Ends of All Pipes Flattened and Covered With Paper and Cheese Cloth
(b) SECTION A-A

FIG. 12.—CONTRACTION-JOINT DETAILS FOR CUSHMAN DAM NO. 1

readily visualized. This would have made an interesting comparative picture, but, after getting the joint detail descriptions translated and making several layouts, it was apparent that individual fads and fancies had so permeated this realm that it would be very difficult to produce typical illustrations which would do justice to dams in general. Hence, it was decided to summarize instead of illustrate, and with this in mind, a few of the outstanding water-stop features of typical dam construction jobs are reproduced in Figs. 11, 12, and 13.

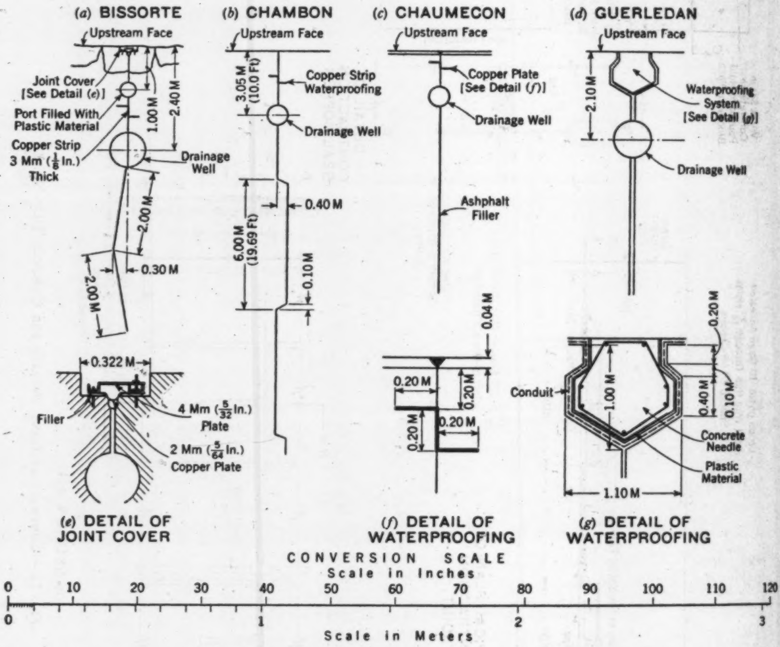
(a) BISSORTE
Upstream Face
Joint Cover [See Detail (e)]
Port Filled With Plastic Material
Copper Strip 3 Mm (1/8 In.) Thick
Drainage Well
2.40 M
1.00 M
2.00 M
0.30 M
2.00 M
(b) CHAMBON
Upstream Face
Copper Strip Waterproofing
3.05 M (10.0 Ft)
Drainage Well
0.40 M
6.00 M (19.69 Ft)
0.10 M
(c) CHAUMECON
Upstream Face
Copper Plate [See Detail (f)]
Drainage Well
Ashphalt Filler
(d) GUERLEDAN
Upstream Face
2.10 M
Waterproofing System [See Detail (g)]
Drainage Well
0.322 M
Filler
4 Mm (5/32 In.) Plate
2 Mm (5/64 In.) Copper Plate
(e) DETAIL OF JOINT COVER
0.04 M
0.20 M
0.20 M
0.20 M
0.20 M
(f) DETAIL OF WATERPROOFING
Conduit
1.00 M
0.20 M
0.40 M
0.10 M
Concrete Needle
Plastic Material
1.10 M
(g) DETAIL OF WATERPROOFING
CONVERSION SCALE
Scale in Inches
0 10 20 30 40 50 60 70 80 90 100 110 120
0 1 2 3
Scale in Meters

FIG. 13.—CONSTRUCTION-JOINT DETAILS FOR FRENCH DAMS

A review of ten papers[50] on joints in dams in Great Britain, France, Italy, Switzerland, Sweden, Germany, and the United States indicates, very conclusively, that there is a preference for some kind of a water stop across all transverse joints near the upstream face, although no writer gives an adequate analysis of the reasons for, or benefits to be derived from, the use of water stops; nor is it recognized that very few holes in the water stops of a dam practically eliminate the structural benefit. Data on water-seal details in various dams are assembled in the Appendix. Copper for water stops seems to be preferred to any other metal by the designers of all countries.

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[50] Question IV, *Transactions*, 2d Cong. on Large Dams, International Commission on Large Dams of the World Power Conference, Supt. of Documents, Washington, D. C., 1938, Vol. III, p. 3 *et seq.*

In commenting on contraction and expansion joints in a power flume, Hugo Flodin (civil engineer at the Royal Board of Waterfalls, in Sweden) states[51] that "to prevent leakage and avoid electrolytic action, lead sheets have been placed at each side of the copper sheeting." This is the only reference, in the aforementioned ten papers on construction joints, to possible electrolytic action in connection with the use of copper for water stops, although it has been known for many years that such action is within the realm of possibility. Just how permanent the copper stop is as a water seal will probably never be known, because few dams will ever be torn down and the water seal examined in detail to determine its qualities of watertightness and durability. The writer is inclined to the opinion that, as a watertight membrane, a copper stop is open to serious question and at best is merely a deterrent to the passage of water through the joint. This should not be taken as a condemnation of metal water stops, but rather as a word of caution against placing too much confidence in the watertight membrane conception of a metal stop and in its permanence, especially where structural sufficiency might later be affected adversely by leakage due to its ultimate deterioration. After all, even if the copper stop remains tight at its many seams, the concrete into which it is set is not watertight, and the path of percolation around the stop is all too short. Of the available methods of limiting objectionable leakage through the joints of a dam, it is believed that metal water stops near the upstream face are least permanent and that where any and all leakage must be prevented consideration should be given to grouting the joints or to closing plug joints (slots). Possibly a joint drainage system, so common in European dams, should be installed.

In the United States, it has become nearly standard practice to use 20-oz soft annealed copper for water stops. In Boulder Dam, however, the water stop selected was monel metal because of its record for durability. In other countries also, copper plate seems to be preferred for nearly all installations where a water stop appears desirable, although there are occasional installations where monel metal or stainless steel is used. The thickness shown on most foreign illustrations available is 2 mm, which is nearly three times the thickness of 20-oz copper (0.027 in.). The shape of the stop and the length provided for embedment, however, are so variable that it is difficult to summarize satisfactorily. Practice in other countries seems to be to place the water stop much farther from the upstream face than is customary in the United States. Most of the installations in the United States are only about 12 in. from the upstream face. It is believed that much more satisfactory and permanent results would be obtained if water stops were set far enough back from the surface to be beyond the zone of appreciable temperature variation, which is undoubtedly detrimental to the bond between the concrete and copper.

In the United States, the **Z**-stop seems to have proved more satisfactory and serviceable in actual use than any other type. This type, however, does not fit all conditions and hence the **M**, the **U**, the **V**, and many other modifications, as illustrated in Figs. 11 and 12, have been developed for special jobs where the joint movements are rather complicated. Whether annealed copper will retain the qualities imparted by the annealing process, permanently, where the joint is subject to constant change in width due to temperature variations, or will ultimately become brittle and crack, is also open to question, especially in joints where the copper stop is merely crimped and the opening and closing of the joint produce an appreciable movement in the crimped bend. Water and grout stops should be jointed by brazing rather than soldering since the latter method has not proved nearly as satisfactory as the former.

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[51] "Contraction and Expansion Joint in Intake Flume," by Hugo Flodin, *Transactions*, 2d Cong. on Large Dams, International Commission on Large Dams of the World Power Conference, Supt. of Documents, Washington, D. C., 1938, Vol. III, p. 188.

Some kind of bead or mechanical bond at the embedded end of the stop seems imperative to assist in preventing bond failure. This is rather difficult to secure if the copper is purchased in rolls and cut to lift length on the job. The same result as with the bead is accomplished, however, if the edge is snipped every few inches and alternate sections bent to form a short L, or if bond holes are punched in the copper near the outside edge. A mechanical bond of some kind is believed more important than increased length of embedment, especially if the difference in coefficients of expansion of copper and concrete is given any weight as affecting the validity of the bond where temperature ranges in the surface concrete surrounding the copper stop may be as much as 50° F annually. One of the construction problems that is difficult to solve satisfactorily is to attain 100% bond in the embedded part because it means eternal vigilance throughout the construction period. Rock pockets and big slugs of concrete always seem to happen at the wrong time and in the wrong place to produce a watertight water stop. Wherever watertight joints in massive structures are necessary, it is believed that joint grouting after minimum volume is reached will give more satisfactory results than dependence on metal stops whose function can be so easily circumvented if rock pockets or poor bond develops around the embedded legs of the water stop—as so often happens, especially on the blind side of the joint.

In Fig. 13 are shown the essential details of various transverse joints that have been used in France and other countries in Europe. This construction appears to have considerable merit from the standpoint of durability and watertightness, and it is believed that it could be used to advantage in reduced section in many places.

Undoubtedly, the metal water stop is a necessary adjunct of the construction joint in thin concrete structures; but its desirability is questioned for massive structures which, once cooled to mean annual temperature throughout, are subjected to no further volume change at depths not affected by annual temperature variations. It would seem that either the grouting of the construction joints or joint construction embodying slots, which seem to be quite popular in foreign countries, is a better, more permanent, and more satisfactory means of rendering a construction joint area tight than dependence on a thin metal strip. Any imperfection in the metal stop, or the concrete adjacent thereto, from any cause whatsoever, exposes the entire joint area to any objectionable effects that water may have on the joint. The outstanding example of a slot or a closing plug with few, if any, objectionable results is the cooling slot in the center of Boulder Dam. It is 8 ft wide and was filled with concrete at a fairly rapid rate after the cooling of the dam was completed. Subsequently, cores drilled across the contact surfaces of the plug and the dam did not develop any reason to question such methods of construction procedure.

GROUT STOPS

The grout stop is an outgrowth of the water stop, its function being the same, but the required period of serviceableness being entirely different. Hence, on some dams, there have been developed, for joint grouting purposes, entirely different types of stops. On Boulder Dam, for instance, the grout stop consisted of a straight copper strip coated with asphalt mastic and embedded across the joint, the mastic serving to maintain grout tightness and still permit opening of the joint without rupture of the metal stop. The kind of metal that should be used in a grout stop is of relatively little importance since permanency is not one of the requirements.

The proper location of a grout stop is a moot question. If the grout film near the surface of the dam was a permanent and durable filler, the closer the grout stop could be located to the surface of the dam the better. However, since the durability of the grout film near the surface appears so questionable, it would appear that in the interests of permanence for the film the grout stop should be set well back from the surface. Temperature variations at 3 ft from the surface would certainly be much more conducive to permanence in a grout film than at 9 in. It is noted that the tendency in foreign countries is to place grout and water stops farther from the face of the dam than in the United States. Grout stops placed too close to the surface give trouble during the grouting operation because the concrete cantilever beam supporting the stop fails, due to grout pressure from the interior of the joint, and a sliver of concrete adjacent to the joint is pushed off.

JOINT FILLERS

The indiscriminate use of joint fillers—a design and construction practice that is on the increase in the United States—is objectionable from the standpoint of appearance; and in many cases there is no basic reason for their use. The necessity for a joint to eliminate an unsightly crack does not carry with it the parallel necessity for some kind of joint filler. These are two entirely different problems—one involving tension and the other compression. Even where a joint filler is indicated as necessary, a relatively thin, inconspicuous filler may answer the purpose just as well, or even better, than the thick one which is such common practice.

Thin slabs and some thin walls require suitable provision for expansion without buckling or spalling if the temperature differential to which they are subjected is rather high and the structure is not restrained. However, there is still too much of a tendency to use thick joint fillers and wide joint spacing, rather than close spacing of joints; and to use the minimum thickness of filler and a slight chamfer, instead of a filler, wherever it will accomplish the desired purpose—that is, prevent spalling at the surface.

During recent years the writer has made a special point of observing and photographing construction joints in concrete structures with a view to determining just where a joint filler is needed and where a chamfer only is indicated. As a result of these observations, it is concluded that many joints that should have spalled, because of the absence of adequate joint treatment, did not, and most of those that did show some spalling did not need a joint filler but should have had a small chamfer only. It was also observed that the fillerless joints were much more pleasing to look at than the wide joints that had big blotches of joint filler protruding.

Fig. 14 shows a vertical construction joint in the parapet on a curved gravity dam which is typical of the entire dam in regard to the result of ten years' exposure to the elements in a moderate climate, ranging from a few degrees below freezing to 110° F. This joint does not contain a filler and was not painted. A half-inch chamfer at this point, as in most other similar joints on both parapets, would have sufficed to prevent what little spalling has occurred. The sun strikes the downstream face of this dam during the greater part of the day, and as a result the crest of the dam at the center deflects upstream every summer ¾ in., regardless of the level of the water in the reservoir; yet there is little or no evidence of surface compression spalling on joints that were not chamfered.

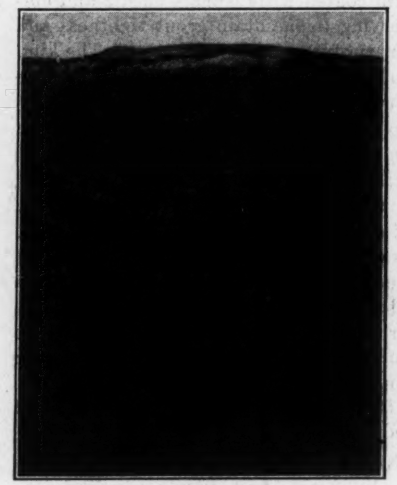

Fig. 14

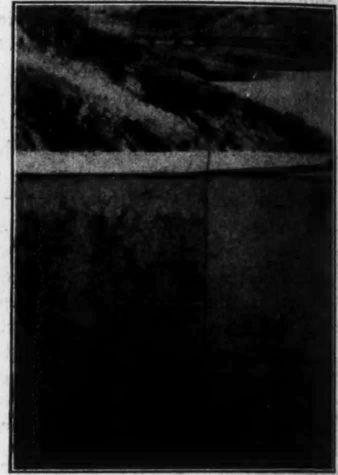

Fig. 15

Fig. 15 is also typical of the joints in the parapet on a straight dam in which the joints were neither treated with a filler nor mastic of any kind; nor were they even painted. Here, too, the result would have been more pleasing to the eyes if a slight chamfer had been used, but there is certainly no indication of the necessity for such joint fillers as is seen in so many modern designs.

Thick straight walls, dams, and thick slabs anchored to rock do not require joint filler if properly chamfered, since in so doing the surface layer of concrete is relieved of stress from contact across the joint and the thrust due to expansion is distributed over sufficient cross-sectional area within the mass to reduce the stress at the surface to limits well below the point of rupture.

In dams and in thick walls and slabs the initial temperature due to heat of setting of the cement will rarely ever be attained thereafter throughout the mass, and hence, if the possible surface spalling at the joint is provided for by proper chamfering, no further consideration need be given to the remainder of the joint surface. Even the painting of the joint to prevent possible bond is unnecessary and is not practiced in dams where joints are to be grouted in the interests of rendering the structure more nearly monolithic. Hence, if not needed for joints that are to be grouted, it certainly is not needed for the average mass concrete joint surface.

JOINT GROUTING

As has been noted previously herein, the successful grouting of the joints in a dam is a highly specialized line of work requiring adequate equipment and a carefully developed and proved technique. The ideal joint-grouting job would be one that would permit the simultaneous grouting of all joints in the dam for their full height. This procedure, however, is rarely possible unless the dam is limited as to height and length. The very nature of the work requires careful advance planning and the checking of every part of the grouting system prior to starting the grouting operations, because, once started, it is difficult, if not impossible, to stop long enough to correct defects or secure additional equipment.

The greatest detriment to successful joint grouting is leakage around or through the grout stops, especially the horizontal runs. A nail hole in the copper, a small break in the soldering or brazing at a joint, or a rock pocket on the blind side of the joint may make it impossible to build up any pressure in the joint or even to fill the joint completely with grout of the proper consistency. This is due primarily to carelessness during the process of installing the grout stop; but the grout stop is just another one of those details that cannot occupy "the center of the stage" if the economics of concrete yardage placement is also striving for the "limelight," as it usually is.

Leakage affects the quality of the resulting grout film, the proper sequence of joint grouting operations, the drainage system, the uplift measurement experimental equipment, if any, and last, but not least, the structural sufficiency of the grout film.

In the grouting of the vertical radial joints in a high arch dam in a narrow canyon in which the grouting operations must be handled in lifts and the grouting of every lift tends to open up a V-shaped opening in the lift below, the horizontal sections of the dam occupied by these V's are prevented from taking their proper proportion of arch stress and consequently their share of the load is shifted elsewhere. Thus it would be possible with considerable abutment yielding during the grouting operation to decrease the bearing area between voussoirs and thus increase the arch stresses to undesirable limits. Hence, it must be realized that grouting joints in dams, to accomplish or even to approach a better distribution of stress than would be obtained in an ungrouted joint, is a highly specialized job that requires experience, a carefully planned method of procedure, an adequate organization, and equipment suitable for that particular class of work.

Cores drilled across grouted joints have demonstrated rather conclusively that grout films of satisfactory density can be secured and also that joints which have reopened may be regrouted successfully; but whether the conception of "monolithicness" popularly held is generally achieved or is even a "fifty-fifty proposition" is open to question. The fragility of the film and its susceptibility to bond rupture, if the joint space is not completely and permanently filled, cannot help but influence any one who analyzes a dam in all its minutiæ of design and construction detail to carefully weigh for future construction the relative value of grout versus slot type of joint construction.

Before starting on the extensive program of joint grouting which the Bureau of Reclamation has done in recent years, a comprehensive series of laboratory tests was conducted with a 5-ft by 7-ft split concrete cylinder in which the width of the joint opening, the consistency of the grout, and the application of pressure were varied through wide limits. As a result of this series of tests, a joint-grouting technique was perfected, which, with the changes and improvements normal to field progress on all such work, has given very satisfactory results as far as all outside appearances are concerned. Whether the joint coverage approaches 100% and whether each joint is completely filled at the time the grouting operations are completed are factors probably open to speculation. That a dense grout film of low water content can be produced with moderate pressures is a fact supported by diamond-drill records and experimental data.

Experience to date seems to indicate rather conclusively that, in order to grout a joint satisfactorily and be able to take care of emergencies when plugged pipes and joints occur, it should be possible to circulate the grout in the joint. The system should also be designed so that it will be possible to weep off thin and foamy slugs of grout mixture and, when the joint has been filled with grout of uniform consistency, to apply a reasonable amount of pressure (25 to 50 lb per sq in.) so as to drive excess water in the grout film into the adjoining concrete and decrease the water-cement ratio of the film, thus increasing its strength and other desirable qualities. With the cement screened through a 200-mesh sieve, the joint-grouting operation generally can be accomplished by starting with a water-cement ratio of 1.0 and reducing as soon as conditions warrant to a ratio of 0.75, more or less. This procedure, of course, assumes proper preparation of the joint prior to grout injection.

The pressure that is applied to drive the surplus water out of the grout film and into the adjacent concrete has a very pronounced effect on the quality of the film and hence should be given as much consideration as is practicable under the pressure limitation dictated by the structure and the site. The flowage properties of the cement grout are quite materially affected by the coarseness of the cement. Cement screened through a 200-mesh screen was used in the grouting of Owyhee and Boulder dams and has a very definite advantage over unscreened cement, since unscreened cement grout causes more or less plugging in the pipes and fittings, will not stay plastic as long as screened cement grout, and will arch in the finer cracks (0.01 in. to 0.03 in.), thus preventing complete coverage of the joint surface. Air separation is now displacing the screen for the production of fine cement for the grouting of contraction joints. The screening of cement for foundation grouting is not ordinarily necessary since poor flowage properties due to cement-particle size can be offset by a little increased pressure. This pressure remedy, however, cannot be applied effectively to joint grouting in dams because of the danger of objectionable displacement and the creation of new and irregular cracks that would be difficult, if not impossible, to grout.

HEIGHT OF LIFT

It will be noted by reference to the Appendix that the height of lifts in concrete dams has varied through rather wide limits with probably more yardage placed in 5-ft lifts than in any other. The 4-ft lift was quite popular at one time in the United States, and then gradually it was displaced with the 5-ft lift, with the result that considerable economy was effected in the placing operation without much additional cost for form work. In recent years there has been considerable agitation for 10-ft lifts in massive construction, the same as in buttressed dams. This move has not proved very satisfactory, and the general result is increased cracking over that obtained in the 5-ft lift. From the construction standpoint it might be possible to perfect satisfactory and economical forms for 10-ft lifts in block or monolithic work; but from the concrete placement standpoint the results are not as satisfactory as with the thinner lifts, and, in addition, the increase in maximum temperature within the mass is decidedly objectionable.

The 5-ft lift appears to be a happy medium and when coupled with a placing schedule of five days between lifts gives reasonably satisfactory results, other things being equal. It is noted that, in the construction of Hiwassee Dam, 2½-ft lifts with five days between pours have been used with considerable success to decrease cracking, in getting off the foundation. Douglas McHenry and Roy W. Carlson have presented[52] some pertinent and interesting data along the general line of the present paper, especially the last sentence of their concluding paragraph which notes that "It has been shown that minor refinements in the conventional design methods which determine the general shape and dimensions of a gravity dam have less effect upon stress distribution and safety than refinements in the construction procedure. This suggests that the construction program might well be considered as the governing element in the design of large gravity dams." How true! Yet how difficult it is for the constructer who is responsible for progress and economics to appreciate it.

It should be noted that of the seventy-eight dams listed in Table 11, sixty-one were completed since 1925 and only seventeen between 1889 and 1925. This ratio is fairly representative of masonry dams 100 ft or higher, it is believed. Hence, inasmuch as so little time has elapsed since quantity production of portland cement concrete dams was started, there is little wonder that the cracking is not under better control.

The outstanding dam of the seventy-eight, in point of high cement content, is the Bleiloch Dam, in Germany, completed in 1931. It should also be noted that the joints are 82 ft apart, the lifts 8 to 9 ft thick, and that cracks extended entirely through the monoliths on about 40-ft centers for the lower portion of the dam where the cement content was as high as 1.57 bbl per yd.

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[52] "Measuring Dam Behavior," by Douglas McHenry and Roy W. Carlson, *Engineering News-Record*, March 30, 1939, p. 440.

The average of all transverse joint spacings that extend to the foundation for the dams listed in Table 11 is 65 ft, with a minimum of 25 ft for Deadwood Dam—a constant radius arch 165 ft high—and a maximum of 150 ft for Arrowrock Dam, in Idaho, and Pardee Dam, in California. The former is an excellent example of what can be accomplished with low cement content relative to the elimination of cracks; and the latter is equally convincing that 1 bbl of cement per yd, 5-ft lifts, and wide spacing of transverse joints on dam sites of that type will surely produce objectionable cracking.

ACKNOWLEDGMENTS

The writer wishes to acknowledge his indebtedness to George M. Tapley, Assoc. M. Am. Soc. C. E., for assistance in assembling and compiling the material for Table 11, and to the many engineers who have made it possible to present such data.

APPENDIX

CONSTRUCTION JOINTS IN CONCRETE DAMS

The data presented in Table 11 may be further explained and, in some cases, modified, as follows:

CONCRETE YARDAGE

Referring to Col. 6, Table 11, the dams that were of cyclopean concrete and stone masonry are further described in Table 12.

HEIGHTS

Three of the dams in Table 11 (Cols. 7 and 8) were increased in height from the value given. For example, Assuan Dam (Item 4) was raised 16 ft in 1912 to a total height of 112 ft. Marshall Ford Dam (Item 49, Table 11) was completed to a height of 190 ft in March, 1939, when the Lower Colorado River Authority announced its decision to complete the high dam immediately to 265 ft. Ruby Dam (Item 66) will be raised, ultimately, to 658 ft.

JOINT SPACING

Arrowrock Dam (*Item 2, Table 11*).—From the top down 70 ft the transverse joints are spaced at 25 ft; from 70 ft to 130 ft, the spacing is 50 ft; and from 130 ft to 215 ft, the spacing is 150 ft.

Ashokan (*Item 3*).—From the top of dam down 110 ft the transverse joints are spaced at 68 ft; and from 110 ft to 200 ft, the spacing is 168 ft. During construction there were transverse cracks to rock at each of the deepest expansion joints, spaced at 168 ft, with intermediate cracks about midway between.

Assuan Dam (*Item 4*).—In the Assuan Dam, a longitudinal slot 6 in. wide was left between the new masonry and the old. This slot was filled with grout after cooling. When this dam was raised (Item 5) the new concrete buttresses on the downstream face were separated from the old masonry by wrought iron, non-corrosive, bearing plates 7 mm thick.

Barberine Dam (*Item 6*).—Cracks occurred near the top of Barberine Dam which were from 0.2 to 0.5 mm wide, penetrating at least 1 m from the surface.

Dams in France.—Details of the construction joints in Bissorte Dam (Item 8), Chambon Dam (Item 18), and Chavanon Dam (Item 19) are shown in Fig. 13.

Black Canyon Dam (*Item 9*).—At Black Canyon Dam the spacing of transverse joints was 75 ft in the spillway section and 50 ft in the abutment sections.

Bleiloch Dam (*Item 10*).—The Bleiloch Dam showed shrinkage cracks at about 40-ft centers, extending from the foundation to 50 ft below the top of the dam and through the entire thickness of the structure.

Bonneville Dam (*Item 11*).—In one monolith two vertical cracks parallel with the axis were found soon after the mass was placed. Each crack started from an abrupt change in bed rock level. The maximum opening was 0.03 in. but later decreased to somewhat less.

Boonton Dam (*Item 12*).—The combined width of all cracks in Boonton Dam was about 3.5 in.,[53] consisting of seventeen main cracks with top widths of $\frac{1}{16}$ in. or more; sixteen small cracks with top widths less than $\frac{1}{16}$ in.; and thirty-three half cracks part way across the top of the dam. Main cracks occur at very regular intervals of 100 ft and no effort has been made to treat any of these cracks or reduce the leakage.

Boulder Dam (*Item 13*).—In Boulder Dam the transverse joints are spaced 60 ft on the axis and vary from a minimum of 25 ft at the downstream toe to a maximum of 66 ft at the upstream face of the dam. The circumferential joints vary from 30 ft near the downstream toe to 50 ft at the upstream toe.

Camarasa Dam (*Item 16*).—From the crest of Camarasa Dam downward 72 ft, the transverse spacing of joints was 52.5 ft, as shown in Table 11, Cols. 10 and 11.

Canning Dam (*Item 17*).—The spacing on Canning Dam is 45 ft from the crest down to 80 ft below the crest. For the lower part of this dam, the spacing is 90 ft.

Claytor Dam (*Item 22*).—In the lower part of a few large monoliths of Claytor Dam there are longitudinal joints about midway between faces. These joints are not grouted; but the transverse contraction joints have an 18-in. strip of expansion joint filler at both upstream and downstream faces. In each horizontal 5-ft lift joint there is a 24-in. by 6-in. raised key, a copper-bearing steel plate water stop and 8-in. by 10-in. drains 3 ft, 5.5 ft, and 7.5 ft, respectively, from the upstream face. To prevent the retention of any inferior material at the top of each lift, a strip 18 in. wide and 2 in. to 5 in. thick (depending on the slump) was removed before the initial set occurred, along all permanently exposed faces. Joints in the intake, the bulkhead, and the spillway sections are spaced on 44-ft, 54-ft, and 60-ft centers.

Cignana Dam (*Item 23*).—The principal joints of Cignana Dam, at 98-ft centers, intersect the entire cross section from crest to foundations. Secondary joints occur from the upstream face to the inspection pit (see Fig. 9).

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[53] "The Effect of Temperature Changes on Masonry," by the late Charles S. Gowen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXI, December, 1908, pp. 391–409, 410, and 413.

TABLE 11.—COMPARATIVE DATA ON

Dam No.	Name	River	Location	Year finished	Type[a]	Volume, in thousands of cubic yards	DIMENSIONS, IN FEET		
							Height Above:		Length
							Foundations	Low water	
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	Ariel	Lewis	Washington	1931	CA	200	313	192	1,300
2	Arrowrock	Boise	Idaho	1916	CG	585	351	....	1,100
3	Ashokan	Esopus	New York	1911	SG	500[h]	252	210	1,000
4	Assuan	Nile	Egypt	1912[e]	SG	704[h]	96[h]	....	6,400
5	Assuan (Raised)	Nile	Egypt	1934	SG	1,730	142	....	6,970
6	Barberine	Barberine	Switzerland	1925	CG	288[h]	259	174	930
7	Big Tujunga	Big Tujunga	California	1931	CA	108	249	....	585
8	Bissorte	Bissorte	France	1935	G	392	196	....	1,780
9	Black Canyon	Payette	Idaho	1924	G	79	184	....	1,134
10	Bleiloch	Saale	Germany	1931	CG	236	213	....	700±
11	Bonneville	Columbia	Ore.-Wash.	1938	SG	500	170	90±	1,250
12	Boonton	Rockaway	New Jersey	1905	G	225[h]	114	110	2,150
13	Boulder	Colorado	Nev.-Ariz.	1935	CG	3,250	726	....	1,282
14	Bull Run	Bull Run	Oregon	1929	CG	222	200	....	1,000
15	Calderwood	Little Tennessee	Tennessee	1930	CA	280	232	200	916
16	Camarasa	Pallaresa	Spain	1920	CG	283	333	....	460
17	Canning	....	Australia	1936	G	....	218	....	1,600
18	Chambon	Romanche	France	1935	SG	392	450	290	1,080
19	Chavanon	Chavanon	France	1925	CG	445	295	....	1,180
20	Cheoah	Little Tennessee	N. Carolina	1919	....	200	225	200	750
21	Chute-a-Caron	Saguenay	Quebec	1931	SG	460	200	170	3,040
22	Claytor	New	Virginia	1939	SG	217	132	125	1,140
23	Cignana	....	Italy	1928	CG	200	183	....	1,320
24	Conchas	South Canadian	New Mexico	1939	SG	750	235	190	1,250
25	Cross River	Cross	New York	1908	SG	161[h]	170	126	987
26	Croton Falls	West Croton	New York	1911	G	260[h]	173	113	1,100
27	Cushman No. 1	Skokomish	Washington	1925	CA	90	275	250	786
28	Cushman No. 2	Skokomish	Washington	1931	CR	....	240	170	580
29	Deadwood	Deadwood	Idaho	1931	CR	55	165	....	750
30	Diablo	Skagit	Washington	1928	CA	330	389	....	1,180
31	Dniepostroy	Dnieper	Russia	1932	G	950	198	153	2,500
32	Don Pedro	....	California	1923	CG	282	288	278	1,040
33	Elephant Butte	Rio Grande	New Mexico	1916	SG	605[h]	306	....	1,155
34	Eguzon	Creuse	France	1927	CG	274	200	....	780
35	Exchequer	Merced	California	1926	CG	369	330	309	955
36	Fifteen Mile Falls	Connecticut	Vermont	1930	SG	....	175	....	....
37	Friant	San Joaquin	California	1943	SG	1,900	300	255	3,430
38	Gibson	Sun	Montana	1930	CR	161	205	....	960
39	Gilboa	Schoharie	New York	1926	SG	....[h]	182	155	1,324
40	Grand Coulee	Columbia	Washington	1940	SG	10,000	540	368	4,140
41	Grimsel	Aar	Switzerland	1931	CG	445	377	328	820
42	Hiwassee	Hiwassee	N. Carolina	1940	SG	770	307.5	....	1,265
43	Hogan	Calaveras	California	1930	CA	122	136.5	113	1,325
44	Hohenwarte	Saale	Germany	1939	CG	608	246	138	....
45	Kensico	....	New York	1916	SG	....[h]	307	168	1,843
46	Laggan	Spean	Scotland	1934	CG	110	186	....	680
47	Madden	Chagres	Panama	1934	SG	525	223	....	974
48	Mareges	Dordogne	France	1935	CA	242	295	....	650
49	Marshall Ford	Colorado	Texas	1939	SG	969	265[h]	....	2,325
50	Martin	Tallapoosa	Alabama	1926	CG	395	168	157	1,300
51	Melones	Stanislaus	California	1926	CR	93	210	....	590
52	Mettur	Canvery	India	1934	SG	2,030	230	....	7,707
53	Montejague	Guadares	Spain	1925	CA	35	273	230	259
54	Morris	San Gabriel	California	1934	CG	450	328	245	756
55	Mount Bold	Onkaparinga	Australia	1937	CR	132	141	130	700
56	Narrows	Yadkin	N. Carolina	1917	CG	525	216	185	1,144
57	New Croton	Croton	New York	1906	SG	855[h]	297	174	1,168
58	Norris	Clinch	Tennessee	1935	SG	1,000	265	....	1,570
59	O'Shaughnessy	Tuolumne	California	1923	CG	398	344.5	226.5	605
60	O'Shaughnessy (Raised)	Tuolumne	California	1938	CG	675	430	312	900
61	Owyhee	Owyhee	Oregon	1932	CG	504	405	....	850
62	Pacoima	Pacoima	California	1927	CA	226	380	355	600
63	Pardee	Mokolumne	California	1929	CG	640	358	350	1,320

CONSTRUCTION JOINTS IN CONCRETE DAMS

JOINT SPACING, IN FEET		Height of lifts, in feet	Hours between pouring lifts	BARRELS OF CEMENT PER CUBIC YARD OF CONCRETE[b]		Water-cement ratio, by weight	DIMENSIONS OF KEYS, IN INCHES	
Transverse (10)	Longitudinal (11)	(12)	(13)	Interior (14)	Exterior (15)	(16)	Size (17)	Spacing, on centers (18)
30±	....	10	....	1.0	....	0.71	54 × 6[d]	...
25, 50, 150	....[h]	4	....	0.96	1.05[b1,h]	....	36 × 10[f]	120
84, 168	....[h]	....	....	1.00	1.30[b2]	....	120 × 12[d]	...
None	....[h]	....	....	....	....[b2]	....	....	...
23	....[h]	....	....	....	....[b2]	....	....[f]	...
82[h]	....	5±	72–120	0.9	....[b2,h]	0.75–0.80	None	...
50	....	....	72	1.05	....	0.73	....	...
49[h]	....	....	....	....	....	....	Broad triangular keys	...
75, 50[h]	....	3.5	72	0.88	....[b2]	....	48 × 9[d]	...
82[h]	....	8 to 9	120–144	1.15 to 1.57[h]	....[b3]	0.69	....	...
60	....	5	72	0.9 to 1.01[h]	1.21 to 1.45[b4]	0.52–0.75	....	...
None[h]	....	....	....	0.68	....[b2]	very wet	....	...
25 to 66	30 to 50[h]	5	72	1.01	....[b3]	0.59	30 × 6[d]	...
40	....	4.5	48	0.81	0.96	0.71	20 × 8[f]	...
50±	....	10	....	1.12	....[b2]	0.67	30 × 8[f]	78
52.5[h]	....	....	....	1.22	....[b1,h]	....	....[f]	...
45, 90[h]	....	6	....	1.10±	1.40±[h]	0.59–0.63	....[f]	...
49[h]	....	....	....	0.67 to 1.0	1.12	0.73	240 × 16±[d]	...
49	....	....	....	....	....	....	240 × 16±[f]	...
50	....	5	....	1.1±	....[b2]	....	30 × 8[f]	78
50 to 55	....	10	....	0.78	1.16[b2]	0.8–0.6	48 × 6[f]	120
44, 54, 60	45[h]	5	72	0.85 to 1.02	120 to 1.35[b5,h]	75, 57[e]	30 × 6[f,g]	120
28, 49, 98	....[h]	....	....	....	....	....	....	...
40 to 50[h]	30 to 50[h]	5	72	0.75 to 0.94	1.00 to 1.19[b5]	0.69–0.62	54 × 12[f]	216
None[h]	....	....	12[h]	0.75	....[b2]	wet	....	...
None[h]	....	....	....	0.65	1.25±[b2]	....	....	...
75	....	5[h]	72–96	1.20±	....[b2]	Slump 4½ in.	....[f]	...
36 to 52[h]	....	5	48	1.25	....[b2]	0.53	96 × 12[d]	...
25	....	4	72	1.32	....[b2]	0.53	36 × 9[f]	...
75	....	5	....	1.27	1.46[h]	0.47	72 × 12[d]	...
50±	40	13	....	....	....	....	....[f]	...
32.5, 65, 130	....[h]	4[h]	24	0.82	....	....	60 × 12[f]	...
100, 50	....[h]	4	....	1.00	1.40[b1,h]	0.65–0.80	18 × 4[f]	...
98	....	....	....	....	....	....	78 × 12[f]	...
25, 50, 75	....	5	....	0.79	richer	0.71–0.8	60 × 12[f]	...
30 to 50[h]	....	....	....	....	....[b2]	....	....	...
50	....	5	72	0.8[h]	1.0[b6]	0.6	30 × 6[d]	...
30, 60[h]	....	4	72	1.0	....[b2,h]	0.67	36 × 9[d]	...
76[h]	....	3 to 6	8–24	1.4	....[b2]	0.41–0.63	....[f]	...
50	50	5	72	1.0	....[b6,h]	0.60	30 × 6[d]	...
25, 49, 98	50[h]	....	....	0.85	1.34	0.83	....	...
38 to 50[h]	....	5[h]	60	0.85	1.2[b3]	0.57	....[f]	...
30 to 40	....	5	....	0.94	....[b2]	0.63	42 × 12[d]	...
49	....	5.9	....	1.28	....[b3,h]	0.67	....[f]	...
73.5 to 79	....[h]	....	....	0.837	....	....	....[f]	...
45	....	3.5	....	1.12	2.0[b2,h]	0.4–0.55	....	...
56	....	5	72	1.00	....[b2]	0.70	36 × 9[d]	...
43	....	....	....	1.13	1.22 to 1.34	0.45	....[f]	...
52	42	5	....	....	....[b3]	....	30 × 6[d]	...
30 to 72	....	10	24	1.00±	....[b7,h]	....	....[f]	...
....	....	4	....	0.78	....	0.75	....	...
126	....	....	....	....[h]	....[b2,h]	....	....	...
40 to 80	....	4	120	0.88	....[b2]	....	....	...
50	....	5[h]	....	0.95	1.10[b3]	0.59–0.54	....	...
40[h]	....	....	....	1.10±	....	0.63–0.67	....[f]	...
50	....	5	....	1.25±	....[b3]	....	30 × 8[f]	78
None[h]	....	....	....	....	....[b2]	....	....	...
56[h]	....	5[h]	72	0.90	1.10 to 1.20[b5,h]	0.65–0.57	....	...
97	....	5	....	1.00	1.25[b3]	....	180[f]	...
			72					
48.5	....[h]	5		1.00	1.19[b2]	0.62–0.58	180[f]	...
50	....	4	72	1.0	1.25[b2]	0.67–0.57	36 × 9[d]	...
50	....	5	....	1.04	....	0.73	....	...
37.5, 75, 150[h]	....[h]	5	....	1.00	....	0.67	....[d]	...

TABLE 11.—

Dam No.	Name	River	Location	Year finished	Type[a]	Volume, in thousands of cubic yards	DIMENSIONS, IN FEET		
							Height Above:		Length
							Foundations	Low water	
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
64	Parker..........	Colorado	Calif.-Ariz.	1939	CR	260	320	....	800
65	Roznow.........	Dunajee	Poland	1939	SG	500	160	....	1,800
66	Ruby...........	Skagit	Washington	1940	CA	....	230[h]	140	490
67	St. Francis......	San Francisquito	California	1926	CG	175	205	....	650
68	San Mateo......	....	California	1889	CG	157.2	154	140	680
69	Santeetlah.......	Cheoah	N. Carolina	1928	CA	195	214	187	1.120
70	Sarrans.........	....	France	1930	G	....	37.	....	710
71	Sautet..........	Drac	France	1934	CA	130	414	....	263
72	Schraeh.........	Waeggital	Switzerland	1925	G	305	362	220	550
73	Seminoe.........	North Platte	Wyoming	1939	CA	161	290	....	540
74	Shasta..........	Sacramento	California	1943	CG	5,400	560	528	2,860
75	Shoshone........	Shoshone	Wyoming	1910	CR	79[h]	238	....	200
76	Tygart..........	Tygart	W. Virginia	1937	SG	1,200	240	...	1,850
77	Waterville.......	Big Pigeon	N. Carolina	1929	CA	123	200	190	790
78	Wilson..........	Tennessee	Alabama	1926	SG	1,240	137	...	4,860

[a] Symbols denote: CA, constant angle; CG, curved gravity; SG, straight gravity; G, gravity; CR,
[1] Sand-cement; [2] portland; [3] low-heat; [4] portland pozzuolona; [5] moderately low heat; [6] moderate and low
[g] 12 on 1, side batter. [h] See modifying comments in the accompanying text.

Conchas Dam (*Item 24*).—The transverse spacing of 40 ft in the spillway section of Conchas Dam was dictated by the spacing of the sluicing conduits. The joints in the non-overflow section are from 40 ft to 50 ft, on centers. Longitudinal joints were provided in the abutment monoliths only, which were placed in blocks in order to maintain the lateral bracing of the abutments during construction. These joints were grouted.

Cross River Dam (*Item 25*).—During the construction of Cross River Dam, four major cracks were observed. They had a combined total width of ¾ in. at the top of the dam and extended 42 ft to 70 ft down the face. Leakage indicated a crack depth of 43 ft, the distance between cracks being 62 ft, 165 ft, and 226 ft.

Croton Falls Dam (*Item 26*).—In the second winter after the construction of Croton Falls Dam three major and several minor cracks appeared. There is considerable seepage every winter, which disappears in summer.

Cushman Dam No. 1 (*Item 27*).—Many horizontal joints showed a slight seepage at first, but sealed themselves in time, leaving the back of the dam covered with calcium deposit.

Cushman Dam No. 2 (*Item 28*).—Some cracking and seepage encountered in Cushman Dam No. 1 were eliminated except for two diagonal cracks at a sharp break in the profile of the foundation and in the base of the left abutment, and one horizontal crack in the thrust block near the top of the right abutment.

Don Pedro Dam (*Item 32*).—From the top of Don Pedro Dam down 62 ft, the transverse joints are spaced at 32.5 ft; from 62 ft down to 127 ft, at 65 ft; and in the remaining height, 130 ft.

(Continued)

Joint Spacing, in Feet		Height of lifts, in feet	Hours between pouring lifts	Barrels of Cement per Cubic Yard of Concrete[b]		Water-cement ratio, by weight	Dimensions of Keys, in Inches	
Transverse (10)	Longitudinal (11)	(12)	(13)	Interior (14)	Exterior (15)	(16)	Size (17)	Spacing, on centers (18)
50	....	5	72	1.00	....[b3]	0.6	30 × 6[d]	...
49	....	....	....	1.12	1.34[b3]	....	....	...
50	....	5	72	1.00	....[b3]	....	60×60×12[d]	..
None[h]	....	....	....	1.12	....[b8]	....	....	...
Irregular blocks	....	....	....	1.23	....[b8,h]	0.44	....	...
50, 40, 37[h]	....	5, 10	....	1.25[h]	1.35[b2,h]	0.64–0.70	30 × 8[f]	78
52	....	....	....	1.0 to 1.3[h]	....[b2]	....	....	...
85±	....	....	....	0.8 to 0.98	0.98 to 1.22	....	....	...
....	....	....	....	0.85 to 1.0	1.34[h]	0.94–0.63	....	...
50	....	5	72	....	....[b6]	....	30 × 6[d]	...
50	50	5	72	1.0	....[b3]	0.6	30 × 6[d]	...
None	....	....	....	1.0±	....	....	....	...
52, 60	....	5, 8, and 10	143, 206, and 212	0.75 to 1.0	1.25[b6]	0.48–0.65	48 × 12[d]	...
50	....	5, 10	....	1.20	....	....	....[f]	...
38, 48, 54[h]	....	4 to 6	....	1.37	1.62[h]	....	24 × 10[f]	...

constant radius. [b] The types of cement, for interior as well as exterior concrete, are identified as follows: heat; [7] slag; and [8] coarse. [c] Interior, 75; exterior, 57. [d] Keyway. [e] First stage completed in 1902. [f] Keys.

Elephant Butte Dam (*Item 33*).—The spacing of the transverse joints of Elephant Butte Dam is 50 ft for the top half and 100 ft for the bottom half.

Fifteen Mile Falls (*Item 36*).—At the end of the first year following the completion of Fifteen Mile Falls Dam there were six of the eight 45-ft spillway monoliths, in which intermediate cracks had developed, and only four of the ten 32-ft monoliths that indicated any cracking. A spacing of somewhat less than 30 ft would possibly have eliminated all cracking. All intermediate cracks start from the foundation and disappear before they reach the top of the dam.

TABLE 12.—DAMS OF CYCLOPEAN CONCRETE AND STONE MASONRY

Dam No.	Name	Plum stone (percentages)	Concrete mix	Facing
3	Ashokan	25	......	Precast concrete blocks
4	Assuan	(Granite masonry with portland cement mortar)		....
6	Barberine	7 to 10	......	Ashlar
12	Boonton	50±	1 : 2.75 : 6.25	Ashlar[c]
25	Cross River	35	......	Concrete blocks
26	Croton Falls	32 to 40	1 : 3 : 6	Concrete blocks
33	Elephant Butte	15±	1 : 2.8 : 5.45	....
39	Gilboa	5.8±	......	Stone masonry
45	Kensico	27	......	....
50	Martin	15 to 20[a]	1 : 3 : 6	Stone masonry[d]
52	Mettur	(Plain concrete; upstream face of rubble masonry, in a specially rich mortar—1 : 2.75)		
57	New Croton	(Rubble masonry, faced with ashlar)		....
75	Shoshone	........[b]	1 : 2.5 : 5	....

[a] Cobbles and plum stones. [b] Clean, sound pieces of granite, hand placed in concrete. [c] Slight leakage when dam was subjected to its full head. [d] Concrete blocks on upstream face and stone masonry on downstream face.

Gibson Dam (*Item 38*).—Gibson Dam has its transverse joints spaced at 30 ft for the top 75 ft of the dam and 60 ft for the bottom part.

Gilboa Dam (*Item 39*).—At Gilboa Dam cracks in two of the monoliths have caused some leakage. However, no consistent cracking has occurred between joints, although the monoliths are 72 ft long.

Grimsel Dam (*Item 41*).—Longitudinal cracks, $\frac{1}{16}$ in. wide and 40 to 60 ft apart, occurred in the base of Grimsel Dam. These cracks were covered with a heavy reinforcing steel mat, and grouted. In subsequent construction longitudinal joints were provided.

Hiwassee Dam (*Item 42*).—The spillway of Hiwassee Dam occupies 260 ft of the central part of the structure. The spacing of joints in this section is 38 ft, with one monolith at the left end 40 ft wide. The joints on both abutments are 50 ft on centers. No cracks have appeared on the faces of the dam and only a few longitudinal cracks between transverse joints.

Kensico Dam (*Item 45*).—At Kensico Dam superficial cracks appeared across the top of the dam between monolith joints. There cracks could not be found in the inspection galleries 6 ft below.

Mount Bold Dam (*Item 55*).—Shrinkage stresses were reported to be rare in Mount Bold Dam. When they occurred they were radial, usually on the upstream face, and then only when the concrete was placed in hot weather.

New Croton Dam (*Item 57*).—In March, 1906, when the New Croton Dam was substantially completed, there were four cracks $\frac{1}{8}$ in. wide and one crack $\frac{1}{4}$ in. wide, as measured in the tunnel 6 ft to 9 ft below the top. Smaller cracks that appeared at coping joints were not visible in the tunnel. Seepage was first observed at two points during the winter of 1938–1939 which disappeared the following summer.

Norris Dam (*Item 58*).—At Norris Dam the spacing of joints is 56 ft throughout, except for the power-house headworks in which there are two monoliths 60 ft wide, one 62.5 ft wide, and one 40 ft wide.

O'Shaughnessy Dam (*Raised*) (*Item 60*).—When O'Shaughnessy Dam was raised, a slot 3.5 ft wide was included to separate the new concrete from the old.

Pardee Dam (*Item 63*).—From the top of Pardee Dam down 79 ft, the transverse joints are spaced at 37.5 ft; from 79 ft to 180 ft down, at 75 ft; and from 180 ft to the foundation, at 150 ft.

St. Francis Dam (*Item 67*).—Contraction cracks at St. Francis Dam occurred in approximately radial planes at 50 ft to 75 ft spacing; but it was concluded that these were not of such a nature as to affect the stability of the structure. The dam failed on March 12, 1928.

Santeetlah Dam (*Item 69*).—The spacing shown in Col. 10, Table 11, for this dam applies to the bulkhead, arch, and spillway, respectively.

Wilson Dam (*Item 78*).—The spacings of 38 ft and 48 ft for the spillway structure were dictated by the bridge pier location; 54 ft for the headworks structure was determined from the spacing of the hydroelectric units.

CEMENT

Table 11 gives 12 hr as the time between pouring separate lifts on Cross River Dam (Item 25). Actually the permissible minimum range was 12 to

16 hr, with a permissible average vertical rise of 18 ft per month. The values shown for Don Pedro Dam (Item 32) were further limited to 40 ft in 30 days. In the Hiwassee Dam (Item 42), 2.5-ft lifts were used on all rock foundations and on concrete that was more than 15 days old. On 15-day concrete, two, and on 30-day concrete, four, 2.5-ft lifts, were required before any single 5-ft lift. The 5-ft lifts of Morris Dam (Item 54) and Norris Dam (Item 58) slope upward 10% and 5%, respectively, in a downstream direction.

Type of Cement.—Special types of cement that could not be described adequately in Table 11 are as follows:

Dam No.	Discussion
2	Portland cement reground with finely ground granite in proportions of 55% cement and 45% granite. Fineness was gaged by a permissible 10% retained on a 200-mesh sieve.
6	Manufactured in revolving tubular kiln, under Swiss specifications.
10	Mix: 34 parts of portland and 66 parts of an admixture, by weight. As indicated in Col. 14, the cement concentration varied from 1.15 bbl at the top of the dam, 1.35 bbl at the mid-section, and 1.57 bbl per cu yd at the base. Note the cracking due to the high cement content in the lower part of the dam.
16	Portland cement reground with crushed sandstone in the proportions of 56% cement and 44% sandstone; fineness standard same as item 2.
22	Both fine and coarse aggregate manufactured from dolomite. Approximately 10% hydraulic lime was used in the interior. The cement content was reduced gradually, because of cracking.
30	For a thickness of 3 ft, along the upstream face of the dam, the cement concentration was 1.46 bbl, as stated in Table 11.
33	Portland cement was reground with crushed sandstone in the proportions of 52% cement and 48% sandstone; fineness standard same as item 2. As shown in Table 11, a somewhat richer mix was used in the upstream face only. The upstream face was also coated with gunite.
37	In addition to the 0.8 bbl of cement, the interior of this dam also contained about 60 lb (1.5 cu ft) of pumicite per cubic yard.
38	To the portland cement an admixture of 1% to 2% diatomaceous earth is added to correct the harshness in the sand when necessary.
40	Although cracking at Grand Coulee Dam was greatly reduced as compared with Owyhee (Item 61) the conditions were not all that might be desired. The decision was made to use low-heat cement to complete the dam. Analysis of crack surveys at three dams of the Bureau of Reclamation

shows the following results in length (feet) of opened cracks per foot of galleries:

Dam No.	Name	Unit length
61	Owyhee	2.47
40	Grand Coulee	1.85
13	Boulder	0.72

44 Portland cement and trass (volcanic tufa) finely pulverized, in the proportions of 60% cement and 40% trass, which delays the setting of concrete and also unites with the lime in cement to form calcium silicate. A patented admixture was used.

46 Fineness standard, 7% retained on a 170-mesh sieve. Referring to Table 11, 2.0 bbl of cement per cubic yard were used in the upstream face of the dam only. Shrinkage cracks were observed on all faces of monoliths, at about 15-in. centers.

50 85% portland and 16% slag cement.

52 80% portland and 20% underburned pulverized brick.

58 In one monolith a slag-cement blend was tried—75% portland and 25% slag cement. Strength and temperature were not changed materially, and shrinkage cracks were still formed.

68 An imported cement, extremely coarse in terms of present-day practice; 10% retained on a sieve having 50 meshes per lin in. The concrete was very dry and was hand-tamped in 3-in. layers. After 50 years of service this dam was found to be in excellent condition.

69 In the gravity section, 125 bbl per cu yd and in the arch section 1.35 bbl.

70 The foundation was of portland cement, 1.0–1.3 bbl per cu yd; and slag cement was used for the upper portion.

71 Two slots, each 4 ft wide, extended through the dam from the upstream to the downstream face, and from the crest to 316 ft below the crest.

72 For the upstream face and in the top 50 ft of the dam, 1.34 bbl per cu yd were used, as shown in Table 11. The downstream face, containing less than 1.0 bbl per cu yd, disintegrated badly and was covered, in 1930, with a stone facing. At an altitude of 3,000 ft, the weather conditions at the site are severe.

78 For all the reinforced concrete, and for the concrete in the dam above the rollway crest, the cement used was 1.62 bbl per cu yd.

Methods of Cooling

A summary of problems concerned with the cooling of concrete is as follows:

Dam No.	Discussion
1	Slots 2 ft wide were left between monoliths, for cooling. River water which was circulated through vertical pipes placed in several monoliths was found to be very effective in reducing temperature. The slots were filled six months after completion.
2	At each joint there was one 5-ft square shaft and two 10-ft square shafts. These wells were filled with concrete during cold weather.
3	Specifications provided one 5-ft by 3-ft well in each joint to be filled with some elastic watertight substance if needed to stop leakage.
4	Longitudinal slots, 6 in. wide, were left between the new masonry and the old. They were filled with grout after cooling.
6	Grouted six months after completion.
7	Provision was made for grouting.
11	Provision was made for grouting.
13	The concrete was first air cooled; later refrigerated water was circulated through 1-in. tubing on 5-ft centers embedded in each lift. After cooling, all joints were grouted in 50-ft lifts.
15	Grouted after completion.
22	Provision was made for grouting the transverse joints only. No provision was made for cooling.
24	Longitudinal joints were provided in abutment monoliths only, the monoliths being placed in blocks in order to maintain the lateral bracing of the abutments during construction. These joints were grouted after cooling.
27	Grout system was never used.
28	Construction details for Cushman Dam No. 2 are shown in Fig. 12. Grout system was never used.
29	Grouted six months after completion.
30	Provision was made for grouting.
37	River water circulating through 1-in. tubing embedded in each lift at 5.0-ft centers near the top of the dam and 2.5-ft centers near the base. Provision was also made for precooling the materials during the summer months.
38	Grouted the winter after completion.
40	River water was circulated through 1-in. metal tubing embedded in each lift. This cooling system was grouted as the construction proceeded.

41 The slats were filled eighteen months after completion.

42 Cooled by refrigerated mixing water; also river water was circulated through 1-in. tubing placed in each lift at points where excessive temperatures were anticipated. In this dam, keys were placed parallel to the downward slope of the dam.

43 Grouted eighteen months after completion.

44 During cold weather river water was circulated through 1.5-in. tubing on 4.92-ft centers, embedded in each lift. During hot weather, refrigerated circulating water was used. No cracks had been observed in the dam to the time of its completion. This cooling system was grouted after serving its purpose.

47 This dam was completed in June, 1934. Reservoir water was pumped through the grouting system during the cool months, until in September, 1936, it was grouted.

49 Provision was made for grouting.

55 Joints were grouted with cement mortar under pressure, after maximum cooling and shrinkage.

59 Joints were grouted in 1938.

60 River water and refrigerated water were circulated through 1-in. tubing spaced 5.5 ft, embedded in each lift. After the new concrete had cooled to 45° F the slot was filled with concrete. When this cooled, the joint between the new mass concrete and the slot concrete was grouted.

61 River water was pumped through the contraction joint grouting system. The joints were then grouted in the spring after the dam was completed. Grouting was done during cold weather.

63 The cooling system was grouted in cold weather. It was grouted a second time in 1932 to reduce seepage and plans were made to grout it a third time if necessary. Some shrinkage cracks, large enough to permit seepage, have occurred.

64 River water, only, was circulated through 1-in. metal tubing embedded in each 5-ft lift, the minimum velocity being 2 ft per sec. This system was grouted after cooling.

66 Provision was made for grouting.

68 Keyed on all sides of each block.

69 Grouted immediately after construction.

71 Two slots, each 4 ft wide, extended through the dam from the upstream to the downstream face and from the crest to 316 ft below the crest. They were filled after cooling.

73 River water was circulated through 1-in. tubing on 5.5-ft centers embedded in each lift. The minimum velocity of flow was 2 ft per sec, and water temperatures varied from

	32° to 56° F. The maximum concrete temperature was estimated at 90° F. Grouted after cooling.
74	River water was circulated through 1-in. tubing on 5.75-ft centers, embedded in each lift. Temperatures of 50° to 90° F are reduced so that volume changes can be completed in time to permit joint grouting as construction proceeds.
76	The bottom 20 ft of each joint were grouted ten months after completion.
77	Slots, 8 ft wide, were filled during cold weather.

COPPER WATER STOPS

The material used for water spouts was predominantly copper, 31 dams reporting the use of this material. The following deserve special comment:

Dam No.	Discussion
7	In Big Tujunga Dam, copper stops were located 12 in. from the upstream face, with galvanized iron grout stops near the downstream face.
14	In Bull Run Dam, 24-in. copper stops were shaped to cover the downstream half of a vertical well, 6 in. in diameter, filled with moist well-tamped clay.
17	Canning Dam had a copper water stop and an asphalt well 8 in. in diameter.
19	In Chavanon Dam there was a flat copper strip upstream from a circular drainage well.
22	In each transverse contraction joint of Claytor Dam, 5.5 ft from the upstream face, there is a V-type stop with 8-in. legs welded to horizontal steel water stops at each 5-ft lift.
24	A 24-in. copper stop in Conchas Dam is shaped to cover the upstream half of a vertical well, 6 in. in diameter, filled with asphalt.
34	In Eguzon Dam, the stops are upstream from a 3-ft square drainage well and,
35	at Exchequer Dam, the stops are upstream from a 10-in. square drainage well.
43	Hogan Dam, similar to Big Tujunga, had V-shaped copper stops upstream and galvanized iron grout stops downstream. The Troiel grout system was used, with 2-in. half round metal forms in the joints.
44	In Hohenwarte Dam a vertical inspection shaft extends the entire height of the dam, in each monolith joint. Downstream from the shafts are vertical and horizontal drain pipes.
45	In Kensico Dam, in addition to copper stops there is also a 4-ft by 5-ft inspection shaft in each joint.
51	Melones Dam has a 10-in. drainage well and copper seal.

54	Morris Dam has copper water seals in 8-in. wells, 2.33 ft from the upstream face.
55	Mount Bold Dam has copper at the upstream face and galvanized iron at the downstream face.
61	At Owyhee Dam there are V-shaped, 20-gage, soft copper stops at both faces, around all gallery openings, and at each 100-ft level.
64	The stops for Parker Dam are flat copper, coated with an asphalt emulsion $\frac{1}{16}$ in. thick.
66	Ruby Dam has V-shaped copper water stops near both upstream and downstream faces.
77	In Waterville Dam there are copper verticals and sheet iron horizontal water stops in each lift.

Other structures reporting copper water stops may be cited without further comment, as follows: Ariel (Z-section), Arrowrock (Z-section), Ashokan, Bissorte (Z-section), Black Canyon, Chambon (Z-section), Deadwood, Don Pedro, Friant, Gibson, Gilboa, Grand Coulee, Madden (V-section), O'Shaughnessy (Dam 59) and Seminoe.

Asphalt Water Stops

The material next in general importance to copper was asphalt, which was reported from ten dams listed in Table 11. These merit special comment as follows:

Dam No.	Discussion
5	The raised section of Assuan Dam was fitted with $\frac{1}{2}$-in. steam pipe for melting the asphalt.
27	Cushman Dam No. 1 has asphalt stops upstream and copper downstream. The steam pipe provided for heating the asphalt has never been used. All horizontal pipes (see Fig. 12) and all vertical pipes (four) have $\frac{1}{4}$-in. by 12-in. slots on both sides of the steam pipe in line with the contraction joint. The slots are 2.0 ft, on centers, and are covered with paper and cheese cloth.
28	Cushman Dam No. 2; same as item 27.
30	In Diablo Dam there is an asphalt stop with a V-shaped copper strip 1 ft wide. A steam pipe was provided for heating the asphalt.
31	In Dnieprostroy Dam the stops are square wells filled with asphalt.
46	Laggan Dam—wells, 6 in. in diameter, filled with asphalt and equipped with copper seals.
52	Mettur Dam—a 12-in. square concrete staunching post coated with asphalt.
26	Pacoima Dam—asphalt wells near the upstream face and copper stops near the downstream face.

69 Santeetlah Dam—6-in. square wells filled with asphalt.

78 At Wilson Dam, joints were filled with tar paper. After leakage developed wells were drilled in each joint and filled with asphalt. The leakage, however, has not stopped completely.

MISCELLANEOUS TYPES OF WATER STOPS

The original Assuan Dam had no water stops. Hiwassee Dam used stainless steel. Other variants from the general rule are as follows:

Material	Remarks
Monel	Boulder Dam has monel water stops and copper grout stops near the upstream face. Galvanized iron grout stops were placed near the downstream face and in all intermediate joints. Tygart Dam has monel **Z**-sections near both faces.
Steel	Calderwood Dam has **V**-shaped, 26-gage steel grout stops in both faces.
Lead	Cheoah Dam and Narrows Dam have **V**-shaped lead strips and an asphalt wedge in each construction joint.
Drains	For water stops, Chute-a-Caron Dam has one well 6 in. square in each joint. Elephant Butte Dam has two open wells, 6 in. in diameter, in each joint.
Jute	Water stops in the Cignana Dam are characterized by an elastic layer of jute and bitumen, protected by a butt joint in reinforced concrete, embedded in the upstream face of the dam.
Galvanized iron	Pardee Dam and the raised section of O'Shaughnessy Dam have galvanized iron grout stops in both faces.
Concrete	In Sarrans Dam there is a reinforced concrete needle at the upstream face and across each monolith joint.

DISCUSSION

William P. Creager,[54] M. Am. Soc. C. E. (by letter).—An excellent résumé of the basic assumptions used in the design of modern masonry dams is presented in the paper by Messrs. Houk and Keener. In engineering, as in most professions, there is seldom exact agreement on many subjects. The writer desires to defend two assumptions of design which differ from those outlined in this paper.

Messrs. Houk and Keener claim that consolidated "silt, sand, or gravel" deposits above concrete dams exert no pressure on the dams. The writer cannot subscribe to that claim. He believes that he needs to present no argument to defend his assertion that coarse "sand and gravel" will exert horizontal pressure no matter how long it has been placed.

As regards the pressure of silt, the authors of this paper, in sole support of their theory, call attention to the Goldbeck cell measurements in the cores of the Miami and Tieton dams to indicate that horizontal pressures did not exceed "hydrostatic pressure" after consolidation had taken place.

Fig. 16 is from data submitted in 1922 by C. H. Paul (6),[55] M. Am. Soc. C. E. A line has been added representing total head of water above any point. It will be noted, as claimed by Messrs. Houk and Keener, that at a depth of 50 ft below water surface, where the material had consolidated, the total pressure of water and soil on the Goldbeck cell was approximately equal to the total head of water above that point.

In order to show that this fact did not prove the lack of soil pressure, it is necessary to investigate the conditions which existed in the core at the time the measurements were taken. These conditions were probably similar to those indicated in Fig. 17, in which the writer has plotted, somewhat diagrammatically, the lines of equal water pressure (not total pressure) in the core of an hydraulic fill dam. The water is seeping from the pool down through the core and into the shell.

It will be noted that, due to the seepage friction losses, the water pressure at any point is much less than that corresponding to the total head above that point. At a point on the center line, where the depth (and hence the total head) is 50 ft, the water pressure is only 30 ft. That is, the water pressure, alone, was only 60% of the total head. Consequently, since the Miami Goldbeck cells recorded pressures equal to about total head, and since the direct water pressure accounts for only 60% of the total head and therefore accounts for only about 60% of the Goldbeck cell reading, the other 40% must have been soil pressure.

The total horizontal unit pressure of soil and water may be computed at any point as follows: Let w = the unit weight of water; p = the ratio of unit water pressure at any point, as shown by Fig. 17, to the total head of water above that point; w_s = the submerged weight of the soil; h = the depth of

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[54] Cons. Engr., Buffalo, N. Y.

[55] See reference (6), bibliography, Fig. 4.

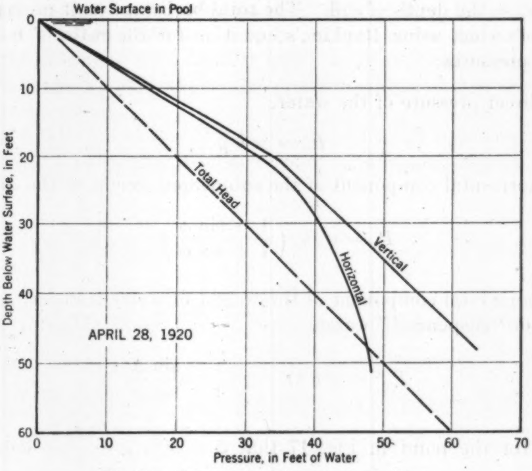

FIG. 16.—PRESSURES IN GERMANTOWN CORE

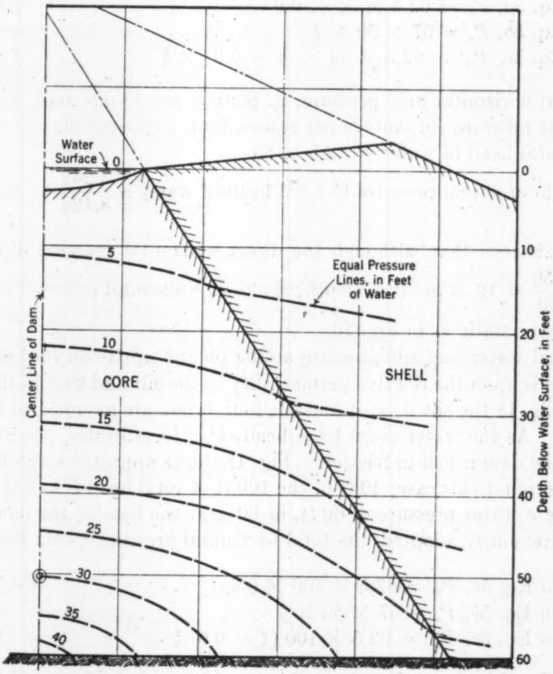
Water Surface
0
Equal Pressure Lines, in Feet of Water
5
10
15
20
25
30
35
40
Center Line of Dam
CORE
SHELL
Depth Below Water Surface, in Feet
0
10
20
30
40
50
60

FIG. 17.—WATER PRESSURE IN THE CORE OF AN HYDRAULIC FILL DAM

water; and h_s = the depth of soil. The total horizontal unit pressure is made up of the following, using Rankine's equation for the ratio of horizontal to vertical soil pressures:

1. The direct pressure of the water,

$$P_w = w\,h\,p \qquad (5a)$$

2. The horizontal component of the submerged weight of the soil,

$$P_s = w_s\,h_s\left(\frac{1 - \sin\phi}{1 + \sin\phi}\right) \qquad (5b)$$

3. The horizontal component of the weight of water transferred to the soil by friction, or "suspended" water,

$$P_w' = w\,h\,(1 - p)\,\frac{1 - \sin\phi}{1 + \sin\phi} \qquad (5c)$$

Assume for the point in Fig. 17 that $\phi = 30°$; $p = \frac{30}{50} = 0.6$; $w = 62.5$; $w_s = 57$; and $h = h_s = 50$.

From Eq. 5a, $P_w = 62.5 \times 50 \times 0.6$ = 1,875
From Eq. 5b, $P_s = 57 \times 50 \times \frac{1}{3}$ = 950
From Eq. 5c, $P_w' = 62.5 \times 50 \times (1 - 0.6) \times \frac{1}{3}$ = 417

Total horizontal unit pressure, in pounds per square foot = 3,242
The unit pressure (in pounds per square foot) corresponding to total head of water = 62.5×50 = 3,125
The ratio of actual pressure to total head of water is $\frac{3{,}242}{3{,}125}$ = 1.037

Thus it is seen that, although the direct horizontal pressure of the soil is 950 lb or $\frac{950}{50}$ = 19 lb per ft of depth, the total horizontal pressure is barely in excess of hydrostatic as in Fig. 16.

The total water and silt pressure acting on the upstream face of concrete dams depends upon the relative permeability of the silt and that of the foundation. In Fig. 18 the silt deposit and the foundation are assumed to be equally permeable. As the water seeps from headwater to tailwater, the entire head acting on the dam is lost in friction. Fig. 18 shows approximate lines of equal water pressure for this case; 40 ft of the 100 ft of total head is lost through the silt, leaving a water pressure of 60 ft, or 60% at the base of the dam. Using the units previously adopted, the total horizontal pressure would then be:

From Eq. 5a, $P_w = 62.5 \times 100 \times 0.6$ = 3,750
From Eq. 5b, $P_s = 57 \times 50 \times \frac{1}{3}$ = 950
From Eq. 5c, $P_w' = 62.5 \times 100\,(1 - 0.6)\,\frac{1}{3}$ = 833

Total horizontal pressure, in pounds per square foot = 5,533

The unit pressure corresponding to total head of water at the base of the dam is 62.5 × 100 = 6,250 lb per sq ft. Therefore the total horizontal pressure of 5,533 lb per sq ft is actually less than that corresponding to the total head. The case is analogous to the pressures in cores of hydraulic fill dams previously mentioned.

It also will be noted that, for this case, the uplift pressure on the base of the dam, instead of varying from headwater to tailwater, as is usually assumed, varies from only 60% of headwater to tailwater.

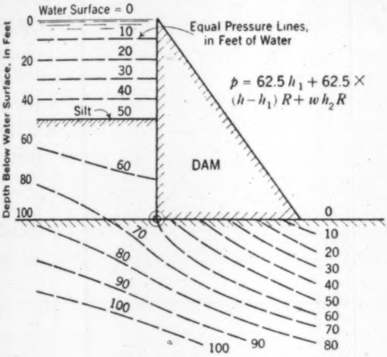
Water Surface = 0
Equal Pressure Lines, in Feet of Water
Depth Below Water Surface, in Feet
0
20
40
60
80
100
10
20
30
40
Silt
50
60
DAM
$p = 62.5 h_1 + 62.5 \times (h - h_1) R + w h_2 R$
0
10
20
30
40
50
60
70
80
90
100
70
80
90
100

FIG. 18.—SILT HAS THE SAME PERMEABILITY AS THE DAM

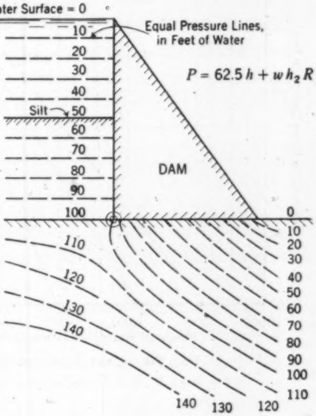
Water Surface = 0
Equal Pressure Lines, in Feet of Water
$P = 62.5 h + w h_2 R$
10
20
30
40
Silt
50
60
70
80
90
100
DAM
0
10
20
30
40
50
60
70
80
90
100
110
110
120
130
140
140
130
120

FIG. 19.—SILT HAS RELATIVELY INFINITE PERMEABILITY

The presence of silt for a case like this would be a distinct advantage. However, the writer believes that the assumed conditions would be very unusual. It is problematical just what allowance could be made for cases where it is felt that the permeability of the silt, in comparison with the permeability of the foundation, is low enough to reduce the pressures on the dam.

In most dams on tight rock foundations, the silt must be assumed much more permeable than the foundation. The typical case of a foundation so impermeable that the friction loss in the silt should be neglected is given in Fig. 19. The conditions are assumed to be exactly the same as in Fig. 18 except that the silt deposit is assumed to be infinitely permeable in comparison with the foundation. Therefore, no head is lost due to seepage through it, and the water pressure at any point on the dam corresponds directly to feet of head on that point.

Using the units previously adopted, the total horizontal pressure (in pounds per square foot) would be:

$$\begin{array}{lrl}
\text{From Eq. } 5a,\ P_w = 62.5 \times 100 \times 1.0 & = & 6{,}250 \\
\text{From Eq. } 5b,\ P_s = 57 \times 50 \times \frac{1}{3} & = & 950 \\
\text{From Eq. } 5c,\ P_w' = 62.5 \times 100\,(1 - 1)\,\frac{1}{3} & = & 0 \\
\hline
\text{Total horizontal pressure} & = & 7{,}200
\end{array}$$

The water pressure is not diminished by the silt, and the silt pressure is appreciable, amounting to 19 lb per ft of silt depth. The writer believes that the permeability of silt may be negligible in comparison with that of most rock foundations, that therefore the pressure of silt is not negligible, and that it must be considered in the design of concrete dams.

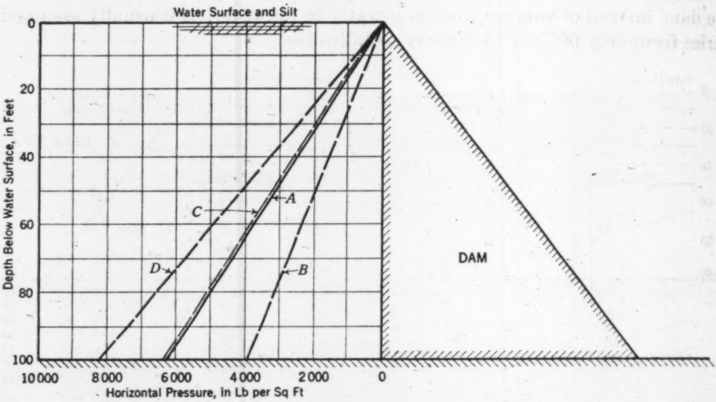

FIG. 20.—UNIT HORIZONTAL PRESSURES IN POUNDS PER SQUARE FOOT
(A = No Silt; B = Foundation Pervious; C = Silt and Foundation Equally Pervious; and D = Silt Pervious)

Fig. 20 indicates the horizontal unit pressures of silt and water under certain reasonable assumed conditions, for the pond completely filled with silt. The differences in pressures are quite marked:

Line A: No silt.
Line B: Foundation very pervious compared with the perviousness of the silt—a very unusual case.
Line C: The foundation and silt are equally pervious—also an unusual case.
Line D: Silt very pervious compared with the perviousness of the foundation—the usual case.

The method for determining the strength of straight gravity dams against sliding, which Messrs. Houk and Keener state has been used "until a few years ago," may be written

$$\text{Factor of safety} = \frac{S_1}{P_h} + \frac{k\,P_v}{P_h} \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

in which $\frac{P_v}{P_h}$ = the ratio of the vertical to horizontal forces—that is, the reciprocal of the "sliding factor" used by Messrs. Houk and Keener; k = the friction coefficient; and S_1 = the usually unknown, and hence neglected, shearing strength developed by keying and bond.

It was specified that, for S_1 equal to zero, the factor of safety must exceed unity.

Messrs. Houk and Keener state that, since 1933, the Bureau of Reclamation has been using a different, or Henny, method; but that method also employs Eq. 6. The sole difference is in the fact that, according to the Bureau method, the factor of safety must be greater than 5 whereas, according to the original method, it must exceed unity when S_1 equals zero. Both of these methods are useful, the Bureau method being adapted to cases where the dam could not possibly slide without shearing and the original method being adapted to cases where the shearing strength is unknown.

In the writer's opinion, the shearing strength of many classes of horizontally bedded rock foundations is unknown since it is impossible to determine the extent of weaknesses in such bedding planes. In such cases the original method should be used and the writer advocates using for k the friction angle of well-dressed specimens. In addition, if the shearing along a weak bedding plane would result in quite a flat surface, a "toe-hold" or some other form of anchorage is necessary.

J. R. Shank,[56] M. Am. Soc. C. E. (by letter).—In the Symposium on "Masonry Dams," the paper by Mr. Tyler is an excellent chapter for a possible comprehensive text on the construction of masonry dams or heavy masonry construction in general. It is comprehensive, concise, and generally well written. No one, save he who has been intimately connected with some large project, can take exception to anything said, and his exception is very likely to arise from some specialty of that particular project. Any one reading this paper will have his own information nicely summarized or averaged, or he will have increased his knowledge considerably.

George R. Rich,[57] M. Am. Soc. C. E. (by letter).—Against the background of their wide experience with the Bureau of Reclamation, Messrs. Houk and Keener have prepared a comprehensive survey of the salient features of modern dam design; and their concluding statement justly emphasizes the complete dependence of all stress analyses upon the one fundamental assumption that the dam is a homogeneous, uniformly elastic structure, free from appreciable shrinkage cracks. Because the presence of these random shrinkage cracks serves to vitiate much of the refinement in computations based upon the elastic theory, and in serious instances to invalidate even the simplest stress determination, reduction and elimination of such defects appear to present the most important current challenge to engineers engaged in dam construction.

Predicated directly upon the primary assumption of an intact homogeneous structure, abundant analytical technique is now available for computing, to a nice degree of refinement, the stresses resulting from the weight of structure, the hydraulic forces, and other readily determined loadings. In fact, development in these particular phases of dam engineering appears to the writer to have far outdistanced much-needed complementary research in the analysis of stresses resulting from setting temperature and shrinkage effects.

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[56] Prof., Civ. Eng., and Asst. Director, Eng. Experiment Station, Ohio State Univ., Columbus, Ohio.

[57] Asst. Chf. Design Engr., TVA, Knoxville, Tenn.

In dealing with the topic of temperature and shrinkage cracks Messrs. Houk and Keener have outlined the important pioneer work of the Bureau of Reclamation in the adoption of low-heat cement, artificially cooling the concrete, constructing massive structures in smaller blocks, and grouting the component units to form a monolithic structure.

The extent to which artificial cooling in dams is economically justified naturally depends upon the size of the project and the progress rate required by the construction schedule. In the case of Boulder Dam it was necessary to pressure-grout the vertical joints between sections to insure development of the requisite horizontal arch action. To be effective, such grouting must obviously be deferred until the major portion of the chemical heat of setting has been dissipated; and to accomplish this dissipation in a concrete mass of such unprecedented size in a reasonable time, it was essential and economical to circulate refrigerated water through an extensive system of cooling pipes distributed throughout the structure.

In constructing the Hiwassee Dam of the Tennessee Valley Authority (TVA) the crack-prevention measures adopted were, in the order of their estimated relative importance: (1) The use of low-heat cement, (2) a slow pouring program, (3) a low proportion of cement in the concrete mix, (4) artificial cooling of concrete after deposition in the forms, and (5) the use of artificially cooled mixing water during the summer.

A basic pouring rate of one 5-ft lift every fifth day was adopted for the dam as a whole, with the stipulation that for a distance of 10 ft above level foundations the rate would be one 2.5-ft lift every three days. Locations where the foundation rock slopes steeply or where large local cavities occur in the foundation are focal points for the development of highly objectionable vertical cracks in planes parallel to the axis of the dam. In such critical locations setting temperatures were controlled by one of two methods—either by a combination of shallow lifts and long exposure (such as a rate of one 2.5-ft lift every eight days) or by circulating river water at natural temperature through a system of pipes embedded in the concrete. Satisfactory temperature control was readily attained by either method and the choice was largely one of economy for each particular case. In rare instances where other than standard lifts were unavoidable, artificial cooling by means of embedded pipes was specified.

The proportion of cement in the concrete mix was initially 0.85 bbl per cu yd, but subsequently it was found feasible to reduce the cement content to 0.80 bbl per cu yd. In summer it was advantageous to cool the mixing water artificially from a natural temperature of about 85° F to 40° F.

The most striking benefit from the use of artificial cooling occurred in a case where it was found necessary to defer pouring certain blocks of the dam to permit passage of the natural flow of the stream during construction, and to carry pouring of the adjacent blocks to a height of about 100 ft. Thus, during setting of the concrete, the block immediately adjacent to the opening was exposed to the atmosphere on one face, while the inshore face was in contact with neighboring recently poured blocks and maintained at an elevated temperature owing to the setting heat of the concrete. On the basis of previous practical field experience, as well as investigations by the theory of elasticity, it was desired to maintain a straight-line temperature gradient through the block to prevent the formation of shrinkage cracks. This was accomplished, readily and inexpensively, by setting resistance thermometers at intervals in the body of the block, pumping natural river water through a system of cooling pipes differentially spaced, and regulating the flow of water through the various sections by means of manually operated valves, so as to approach the desired straight-line temperature gradient. The results were generally very satisfactory, and cracking in the exposed face kept within satisfactory limits.

Since the elimination of shrinkage cracks is becoming recognized as the outstanding difficulty of dam construction, it appears essential as a general principle to provide facilities for eliminating shrinkage defects to the full extent found economically justifiable for the particular project.

With respect to loading conditions, the writer shares the opinion of Messrs. Houk and Keener that advances in earthquake analysis constitute the major current development. However, as a result of recent studies involving impounding structures of widely varying types for the TVA, he is convinced that more consistent results and greater economy, without sacrificing safety, will be obtained by predicating the analysis upon a properly selected period and amplitude for the ground motion and calculating the extent to which the response of the structure approaches resonance, rather than by applying a more or less arbitrary lateral loading of 0.1 gravity.

Instrumental observations by the U. S. Coast and Geodetic Survey indicate that the maximum amplitude to be expected in rock foundations in the United States is about 0.25 in., and that the principal damage results from foundation movements having amplitudes of this magnitude in combination with periods in the order of about 1 sec. Substitution of these values in the basic simple harmonic-motion equation will give an acceleration of about 0.03 gravity, which agrees with the instrumental records.

Recorded movements in rock foundations having high accelerations in the order of 0.3 gravity in combination with periods of 0.1 sec were found to have caused no appreciable damage. One of the reasons for this will be apparent from again substituting these values of acceleration and period in the simple harmonic-motion equation. The corresponding foundation amplitude will be found to be only 0.03 in. Since the natural period of vibration of many typical massive gravity dams will be found to be in the order of 0.10 sec, the question naturally arises why resonance effects do not prove more disastrous in the high acceleration, low frequency brackets.

The explanation follows readily from a study of the standard equation for forced vibrations with viscous damping—that is, with damping proportional to the first power of the velocity and caused by the internal friction of the concrete of the dam. It will be found that damping has a marked influence upon amplitudes in cases where the resonance ratio approaches unity, and only an incidental effect for other resonance ratios. In addition, it will be noted that, for even the condition of theoretically perfect resonance, the magnification factor, which measures the ratio of dynamic to static deflection, will seldom exceed a small finite value in the order of 2.0 for the short natural periods and damping coefficients applicable to ordinary gravity dams.

There is a second factor that will explain why actual magnifications in the high acceleration, short period range do not even remotely approach theoretical damped resonating values—namely, the fact that earthquake vibrations are transitory and that the particular precise forced period required for sharp tuning or resonance is repeated for only a few cycles. Possibly many cycles would be required to build up the amplitude of the structure to the theoretical damped maximum.

Having accepted a period of 1 sec and an amplitude in foundation rock of 0.25 in. as the most rational general basis for earthquake-resistant design, it is interesting to investigate the typical range of standard dams by dynamic methods. Since the natural period of such dams is in the order of 0.10 sec, the resonance ratio, and consequently the magnification factor, will be small and the acceleration of all parts of the dam will be essentially the same as the foundation acceleration—namely, 0.03 gravity. It appears, therefore, that in the United States gravity dams of average proportions would have ample margin of safety against earthquake disturbance if designed for an equivalent inertia loading of 0.05 instead of 0.10 gravity, and since this same acceleration enters directly into the Westergaard formula (15) commonly used for determining reservoir superpressures resulting from earthquake shocks, a proportionate reduction should be affected in the lateral hydraulic loading.

It is in dealing with lighter, more flexible types of hydraulic structures, such as arch dams, power-station substructures, buttress dams, and high spillway piers, that the dynamic method of calculation is most superior to the arbitrary inertia load method. The greater the flexibility of the structure, the more pronounced will be the error of assuming that all parts of the structure have an acceleration equal at all instants to the acceleration of the rock foundation. For flexible structures the use of the dynamic method is essential to obtain deflection curves that do not conflict with common observation and experience.

For reinforced-concrete structures the advantages of the dynamic method of analysis are most marked. Before drawing conclusions in any particular case, it is advisable to check the first and second harmonics as well as the fundamental mode of free vibration. However, as a general rule it will be found that an appreciable saving in reinforcement will be realized; that the reinforcement provided will be distributed more uniformly throughout the length of the structural members; and, most important of all, that the steel will be rationally distributed to sustain earthquake shocks.

It is suggested that dams and related structures on rock foundations in the United States be designed for a foundation amplitude of 0.25 in. and a period of 1 sec, and that such structures be proportioned by the dynamic method based upon the ratio between the assumed forced period of vibration of the foundation and the natural period of the dam.

Mr. Lieurance has prepared a very informative paper that may well serve as a working manual for the design of arch dams. The tables represent an enormous amount of labor, and in most cases will be entirely adequate for preparing the final design.

With respect to articulations or flexible joints in arch dams, the writer is pleased to endorse Mr. Lieurance's opinion that it is undesirable to sacrifice any part of the structural strength of the dam in order to simplify its analysis. Following general practice for similar heavy concrete structures, it is the firm belief of the writer that the most dependable arch dam, conforming most closely in the finished and loaded condition to the design computations, will be obtained by avoiding all articulations and by pressure-grouting the radial joints to full and complete bearing after the major portion of the chemical heat of setting has been dissipated. In the case of large structures, such as Boulder Dam, it will be found economical to dissipate this heat by artificial cooling to meet the requirements of the construction progress schedule. To compensate for residual undissipated heat remaining in the concrete, or for any necessary similar adjustment, the arch dam may be deflected upstream any practicable predetermined amount by selecting the proper grouting pressure, so that the completed dam under its combined loading will be a monolith conforming to the elastic computations and having optimum stress distribution.

Mr. Lieurance has given a clear exposition of the various critical loading conditions that measure the adequacy of design at the abutments upon which the integrity of the arch elements is dependent. The effect of building the upper portion of the dam to greater than normal width to accommodate a roadway may be inferred from a study of any typical hydrostatic load division diagram. In representative cases the upper arch rings carry not only the entire local hydrostatic load, but, because of their superior stiffness, also serve as partial top supports for the vertical cantilever elements, so that the diagram of load tributary to the cantilever elements shows a change in algebraic sign at a point about two thirds the height of the dam. Thickening the upper arch rings aggravates this effect and, in addition, through the medium of tangential shearing stresses, tends to restrain deflection of the adjacent lower arch elements, thus further increasing transfer of load horizontally to the abutments at the higher levels.

With respect to the selection of the economic central angle, Mr. Lieurance cautions against empirical rules, and stresses the desirability of establishing this feature to conform with abutment and foundation conditions of the particular project. The suggestion that the first preliminary examination of stresses be made on an arch section at about two thirds the height of structure again follows readily from an inspection of the hydrostatic load distribution diagram. It will be recalled that in most instances, because of the excess stiffness of the top arches as compared with the top of the vertical cantilevers, the load on the vertical cantilever elements changes from downstream to upstream in direction at about two thirds the height of dam, so that at this point the entire hydrostatic load is carried by the arch elements.

The tables have been so well prepared that there is little opportunity to do more than remark that their use is restricted to symmetrical circular arches and foundation conditions free from singularities.

In investigating thick arches of relatively sharp curvature, it may be desirable to take account of the fact that the neutral axis of the arch does not coincide with the gravity axis. This may be accomplished readily by the well-known Cain-Jakobsen formulas (32).

The tables presented by Mr. Lieurance may be used directly in the determination of earthquake stresses by means of conventional equivalent inertia loadings. Although much research will be necessary to develop the requisite technique, it is suggested that future pioneering be directed toward adapting the dynamic theory of earthquake-resistant design to arch dams. In view of the refinements attempted in connection with foundation deformations, torsion, tangential shear, and Poisson's ratio effects, it appears inconsistent to predicate earthquake stresses, which in many instances may be of comparable magnitude, upon a theory that is merely a convenient fiction when applied to flexible structures in which the magnification factor exceeds unity. The dynamic theory of vibration affords rational solutions for both the longitudinal and transverse forced vibration of fixed-ended beams. It is anticipated that eventually such solutions may be adapted to cover the case of earthquake shocks in directions either parallel, or at right angles, to the axis of the arch dam. Should the mathematical analysis prove cumbersome, valuable assistance might be obtained from shaking-table tests. For actual design the final results should be expressed in simple, convenient form, possibly in the form of uniform loads that would vary for different structural proportions; but the basis for the loading should be rational. It should be postulated upon the critical destructive range of periods and amplitudes of the ground motion at the site, and should involve the magnification factor of the structure or, in other words, the measure of the degree in which the particular structure may approach resonance. This method is attaining recognition as the only satisfactory approach to earthquake-resistant building design and should logically be extended to include hydraulic structures.

The familiar Westergaard formula (15) for hydraulic superpressures during earthquake shocks should be reconsidered concurrently and possibly modified in the case of the thinner arch dams. As is well known, the original theory is based upon the assumption that the magnification factor for the dam is unity, or, in other words, that all parts of the arch dam have the same acceleration as the foundation rock. This assumption is probably sufficiently correct in the case of typical gravity sections, but for flexible arches some corrections may be necessary. Incidentally, any engineer who uses the Westergaard formula should not fail to read the discussion of the Westergaard paper by Boris A. Bakhmeteff, M. Am. Soc. C. E. Professor Bakhmeteff develops the equation against the familiar background of the theory of water hammer in pipe lines, and his clear presentation of the analogy in the case of negative wave reflections has frequently helped the writer to avoid misapplication of the Westergaard expression.

The basic mathematical research for the foregoing investigations will quite possibly be very complicated and laborious; but the final result, to be of any use in practical designing, must be simple and readily usable. The design tables given by Mr. Lieurance are an admirable example of the reduction of extremely involved analysis to simple workable form. It is hoped that ultimately earthquake analysis for arch dams may be rationalized and presented in similar tabular form.

In conclusion it should be remembered that in spite of the powerful methods available there still remain numerous possible combinations of effects produced by thermal stresses, plastic flow, and water soakage that must be covered, for the present, by a liberal margin of safety, and, finally, as stated by Mr. Lieurance, that designs to meet unusual conditions should be undertaken only by engineers having wide experience with arch dams.

ROBERT A. SUTHERLAND,[58] ASSOC. M. AM. SOC. C. E. (by letter).—An extremely valuable addition to the literature on the subject of arch dams is afforded by Mr. Lieurance's paper, and the thanks of the profession are due to the author, especially for the excellent tables which accompany it. These tables enable considerable work in the design of an arch dam to be accomplished with a minimum of fatigue and in a minimum of time.

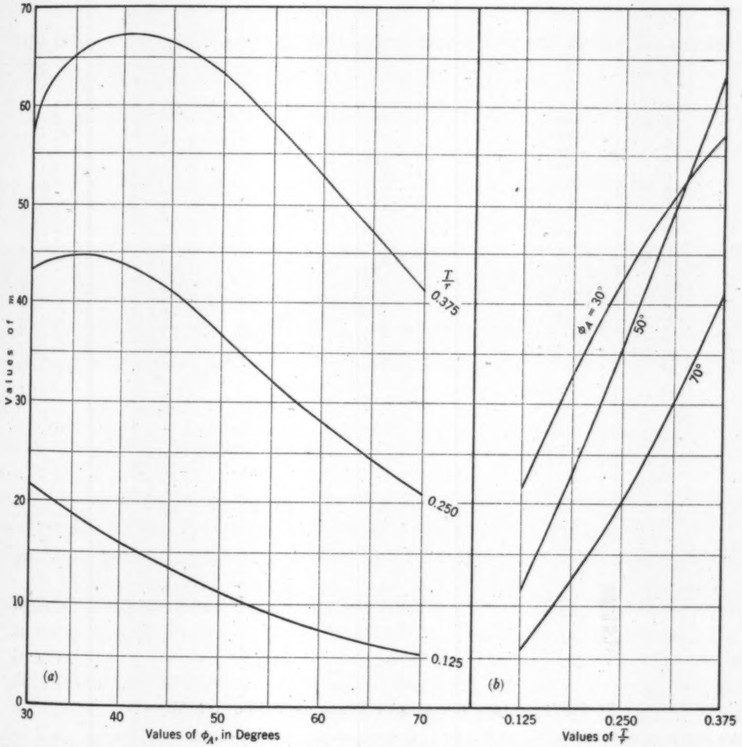

FIG. 21

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[58] Designing Engr., Harza Eng. Co., Chicago, Ill.

The method of interpolation suggested by Mr. Lieurance for finding values intermediate between tabular values seems to be substantially correct for most cases arising in practice. However, a warning should perhaps be added that in certain cases interpolation may give incorrect values, as may be seen by inspection of Fig. 21, which shows a plot of a few values. This difficulty may be readily overcome by interpolating by means of curves covering the range of values which depart appreciably from a linear variation.

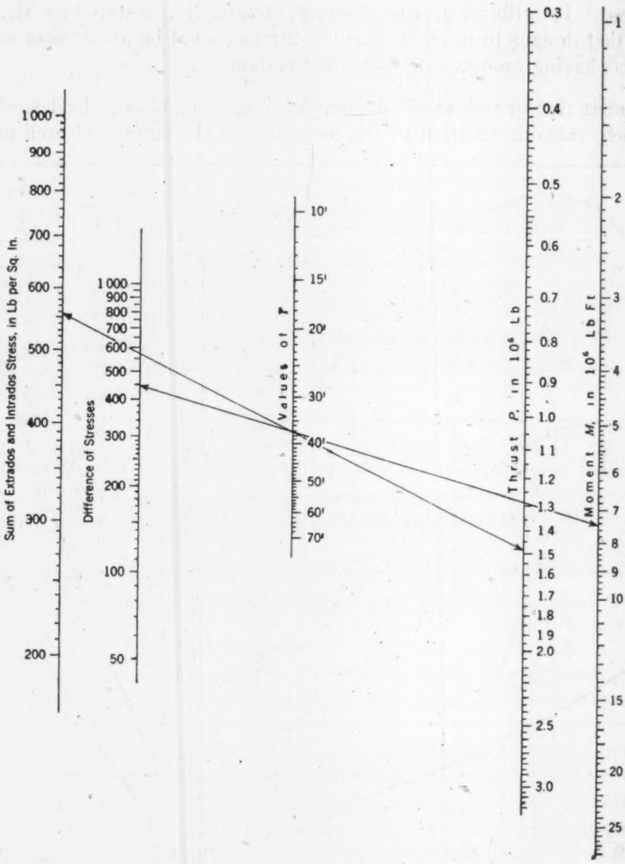

Fig. 22.—Arch Stresses in Terms of Thrust and Moment

The shortening of labor in computation to which Mr. Lieurance has so notably contributed may be extended somewhat further. A simple alinement diagram such as Fig. 22 may be constructed to suit the requirements of the individual user and to facilitate the computation of two quantities—first, the sum of the stresses at extrados and intrados, and secondly, the difference of these stresses. The sum of these stresses is proportional to the thrust, and the difference is proportional to the moment, on the usual assumption of linear stress variation. Hence, by adding the two quantities so found, and dividing by two, one stress may be found; and by subtracting the second quantity from the first and dividing by two the other stress may be found.

Alternatively, Tables 3 to 9 could be extended to give stresses directly for the various loadings, angles, and ratio of thickness to radius as tabulated by Mr. Lieurance. The deflection tables may also be extended to express deflections as a percentage of radius, if a certain modulus of elasticity is assumed, which may reasonably be done. For many purposes, such tables are valuable in giving stresses directly, with the absolute minimum of labor.

Several years ago the writer determined values of stresses resulting from the Guidi formulas,[59] and curves representing a few values are submitted as Fig. 23. Stresses determined from these curves have been checked repeatedly

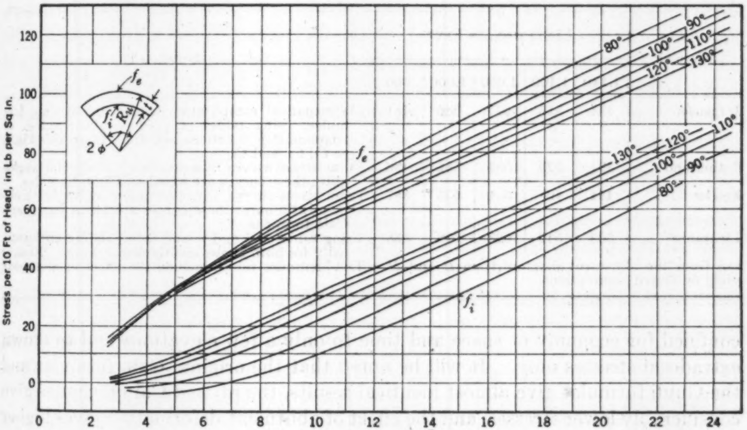

FIG. 23.—CROWN STRESSES IN CIRCULAR ARCH WITH UNIFORM RADIAL LOAD FROM GUIDI FORMULAS

against stresses computed from the original Cain formulas and have been found to check exactly in most cases. Extensive curves based on the revised Cain formulas have been published by Frederick H. Fowler,[60] M. Am. Soc. C. E. Similar curves could be computed readily from Mr. Lieurance's tables, in which abutment deformation is taken account of (which is not the case in the Guidi or Cain formulas), and deflection curves giving deflection in terms of radius could be computed also. Using such curves, it would be possible by a suitable combination of loadings to translate various forms of effective arch load directly into arch stresses and deflections. The writer appreciates particularly the deflection tables, as they readily enable the deflections for many forms of non-uniform loading to be obtained with a fraction of the effort required if one makes use of influence lines.

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[59] *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1623.

[60] *Loc. cit.*, Vol. 92 (1928), p. 1512.

A comparison will now be made between stresses obtained, first, by direct computation from the original Cain formulas, second, from Fig. 23, third, from Mr. Fowler's curves based on the revised Cain formulas, and last from Mr. Lieurance's tables. In the latter case use will be made of Fig. 22 as a time-saving device. To the writer's knowledge, the direct computation from the Cain formulas, even when using previously prepared tables of certain trigonometrical functions, is a matter of two or three hours' work for each elevation. The other methods require minutes only. For the sake of example, use is made of a tabulation[61] of arch stresses in the Diablo Dam, to which reference can be made, if desired. Table 13 shows a comparison of the four methods, confined for economy of space and time to only a few elevations and to crown extradosal stresses only. It will be noted that the original Cain formulas and the Guidi formulas give almost identical results, the revised Cain formulas give considerably lower stresses, and the effect of abutment deformation gives higher stresses. Comparing only the Fowler and Lieurance results, as being based on the most up-to-date data of their respective classes (without and with abutment deformation), it is seen that in the example cited the crown extradosal stress is increased by abutment deformation by from 22 to 151 lb per sq in. This emphasizes the absolute necessity of taking account of abutment deformation, and the tables provided by Mr. Lieurance give a very valuable means of doing this.

TABLE 13.—COMPARISON OF CROWN EXTRADOSAL STRESSES AT A FEW ELEVATIONS IN DIABLO DAM BY DIFFERENT METHODS OF COMPUTATION

Method	ELEVATION, IN FEET					Notes
	1,140	1,100	1,060	1,000	960	
Jorgensen	494	525	577	586	481	Mathematical computation from original Cain formulas, using tables of predetermined values of trigonometric functions noted on Jorgensen's Fig. 4 as C_1, C_2, and C_3.
Sutherland	491	525	584	582	480	From stress curves interpolating visually for angles only; these curves are based on Guidi formulas.
Fowler	481	494	541	533	387	From stress curves, interpolating visually for thickness ratio only; these curves are based on revised Cain formulas.
Lieurance	503	542	623	618	538	Using tables of *h* and *m* and interpolating arithmetically for both angles and thickness ratios. Stresses are from the alinement diagram from *H* and *M*. The Lieurance tables include the effect of abutment yield on stated assumptions.

ROSS M. RIEGEL,[62] M. AM. SOC. C. E. (by letter).—Some aspects of the paper by Messrs. Paul and Jacobs are, perhaps, entitled to extra emphasis. The writer feels that one aim of the process of foundation treatment should be to develop shearing strength, as well as bearing strength, at and below the contact. He sees the quality of resistance to lateral movement as a function of shearing strength in the foundation and in the concrete placed upon it. For this reason he regards the shear-friction factor used by the Bureau of Reclamation and others as representing most truly the capacity to resist movement when proper constants are used in determining the factor.

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[61] *Engineering*, London, July 7, 1933, pp. 2–6.

[62] Head Civ. Engr., Design Dept., TVA, Knoxville, Tenn.

The foundation treatment should be designed to secure shearing strength at the contact. Equally the same quality must be secured in the foundation itself. The process of cleaning out and grouting horizontal seams, referred to by Messrs. Paul and Jacobs, increases the shearing strength of the foundations, as well as the bearing strength, and finds justification therein. Other devices, such as anchor walls and deep seating of the structure, are aimed at the utilization of shearing strength to develop resistance against lateral movement.

Messrs. Paul and Jacobs refer to the interpretation of borings and the necessity of proper interpretation, particularly in the matter of the parts of the record usually labeled "no core." The most competent and continuous supervision possible should be a feature of dam site exploration, so that the significance of core losses can be determined with the greatest certainty. The engineering geologist on the job is worth two or three such men looking over cores when the record is "cold"; and it should be the function of this man to ascertain the reason and significance of loss of core as and when it occurs.

Frequently, the loss of core may be avoided, however, by improved processes. The writer recalls a large number of 2-in. diamond-drill cores from a shale dam site in West Virginia in which the loss of core from apparently sound shale was as great as 20%. Later, a 5-in. diamond drill was installed, and full recovery secured. Incidentally, these cores were preserved under water after removal from the hole, and thus disintegration in the air, a common phenomenon with such rock, was prevented. The writer deems it justifiable to strive constantly for ever larger sizes of core, not because the recovered rock is more significant, but because the gaps require less explanation or speculation.

The tendency to larger holes has brought about the 36-in. and 40-in. drills described by Messrs. Paul and Jacobs. For the study of the actual structure of a foundation, these are most valuable tools. The direct evidence afforded by an inspection of the interior of such a hole is convincing and authoritative. The writer recalls a very expensive shaft excavated into rock for exploratory purposes in which the desired results could have been achieved by the use of one of these machines at a fraction of the cost; but the machine was not available. The Tennessee Valley Authority used one of these drills at Norris Dam,[63] and the results were so useful that similar machines are now used on all projects.

In conclusion, Messrs. Paul and Jacobs are to be complimented on the manner in which they have covered the subject. Their paper is a most valuable résumé of the essentials of foundation treatment for dams.

PAUL BAUMANN,[64] M. AM. SOC. C. E. (by letter).—Mr. Steele's paper on "Construction Joints" is perhaps the most complete one so far presented on this subject. It is based generally on the opinions of engineering authorities from the four corners of the earth and particularly on the wealth of experience acquired by the U. S. Reclamation Service in the course of the design, construction, operation, and maintenance of a great number of major dams of various types. Among these Boulder Dam, with its majestic height of 726 ft above its foundation, is unlikely to be infringed upon for a long time to come. In addition to the wide experience upon which this paper is based, its value is enhanced by its broad and open-minded presentation, all of which render its discussion difficult indeed.

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[63] *Proceedings*, Am. Soc. C. E., March, 1940, p. 385.

[64] Senior Asst. Chf. Engr., Los Angeles County Flood Control Dist., Los Angeles, Calif.

The problem of construction joints probably is as old as masonry construction, but with the introduction of concrete, and particularly mass concrete, it became acute. Its solution is up to the art rather than the science of engineering. It is too complex to be expressed in simple, differential equations such as Airy derived for the equilibrium of stresses in a structure. Approximate solutions for joint spacing may be obtained. However, they are of doubtful value. For example, a block between two joints may be considered as a monolith which is fully restrained from contracting at the base but progressively free to do so with increasing height. The block is assumed to be split along its median plane between the joints due to a crack, the width of which is zero at the base and equal to the full contraction due to a drop in temperature at the top. This drop in temperature must be the equivalent of all volume changes. The forces which may be applied to close the crack are governed by the allowable tensile stress. If the latter is assumed to be uniformly distributed along the surfaces of the crack, then it follows that the joint spacing is directly proportional to the height of the block. According to the evaluation of the modulus of elasticity, the coefficient of thermal expansion, the drop in temperature, and the allowable tensile stress, the proportionality factor varies from about 0.6 to 1.2 and therefore would lead to wider joint spacings, especially for high dams, than are generally used.

On the other hand, if the assumption is made that the tensile stress necessary to close the crack is not uniformly distributed but is concentrated within a relatively shallow zone below the top—for example, within 5% of the height of the block—then it again follows that the joint spacing is directly proportional to the height, but the factor will vary from about 0.4 to 0.7 for values corresponding to those used before. The latter assumption is believed to approximate the physical phenomena more closely, especially in view of surface cooling, than the former. It too, however, is a crude approach to the problem as it does not account for some of the forces which are generated in the still rather mysterious setting process of concrete.

Although the attention given to the spacing of joints normal to the axis has often been wanting, it has been even more so in regard to joints parallel to the axis. Although cracks normal to the axis are not significant from a structural point of view with most types of dams, cracks parallel to the axis are very much so indeed. Such cracks have been observed in straight and curved mass concrete dams as well as in the buttresses of articulated dams.

In one case concerning a mass concrete dam in Southern California, two nearly vertical cracks parallel to the axis formed during construction, dividing a block approximately 200 ft thick, measured upstream and downstream, in three nearly equal parts. This particular block had been left low relative to the remainder of the dam to serve as a spillway during construction. Its average height was about 40 ft. A permanent and a temporary gallery crossed the block in an upstream and downstream direction. They facilitated the observation of the cracks.

Before proceeding with this lift, a mat of reinforcing steel was placed on the then existing top. It was designed to take the full shear which would develop due to full load in the plane of the cracks between the base and the mat. This was done in spite of the fact that computations indicated the closing of the cracks upon completion of the lift due to lateral expansion as expressed by Poisson's ratio. Observations in the permanent gallery substantially confirmed these computations.

Longitudinal cracks, vertical and inclined, were observed in the remaining block of the St. Francis Dam after its failure. Some of them drained water stored inside the block for several days. Whether or not all or some of them were caused by the failure, or were there before, is difficult to decide. The inclined ones near the downstream slope might well have been due to overstress at the time of the failure.

An interesting example of vertical or nearly vertical cracks occurred in the buttresses of a multiple arch dam in Arizona. These buttresses are of the hollow type as described by the late Fred A. Noetzli,[65] M. Am. Soc. C. E. The cracks were between 25 and 50 ft apart. They caused a structural weakness, particularly due to preventing the transmission of shear. To remedy this weakness five horizontal ties were installed in the buttresses of maximum height and a lesser number in the lower ones. The vertical spacing of these ties varied but averaged 40 ft. These ties consisted of a suitable number of heavy railroad rails fastened to steel yokes at the downstream face and to anchors embedded in a heavily reinforced-concrete slab immediately below the upstream or water face. All of the rails in one and the same buttress were equipped with hydraulic jacks and were pre-stressed simultaneously to from 8,000 to 12,000 lb per sq in.

This pre-stress served to cinch up the buttresses, closing the cracks and creating an initial compressive stress of about 30 lb per sq in. It was done when the reservoir was low. After completion of the pre-stressing, the rails were welded together and embedded in gunite which was bonded to the side wall. Continuous floors inside the buttresses were thereby formed.

In the design of the Big Dalton Multiple Arch Dam of the Los Angeles County Flood Control District, inclined joints were provided in the hollow buttresses. The joints follow approximately the direction of the first principal stress and are an average 50 ft apart, measured in a normal direction. Nearly vertical cracks formed in some of the buttresses. They were probably prompted by abrupt changes in the slope of the foundations, where they originated. Fortunately they are too small to be structurally significant. None of these cracks extends across the lowest joint and no cracks formed above the lowest joint. Hence, it appears as if the joints were effective and well worth the extra expense they caused.

Mr. Steele suggests the desirability of turning transverse joints from vertical to normal to the abutment walls, but points out certain practical difficulties in its accomplishment. These difficulties, particularly in regard to form work, no doubt exist and such joints, although effective, would result in slower progress than could be maintained without them. The writer believes that the shape of the abutments has much to do with cracks normal thereto. Hence, in giving due attention to the configuration of the final abutment excavation, and particularly by avoiding abrupt changes in slope as well as by avoiding convexity, most of the causes of these cracks may be eliminated. If these features cannot be provided economically, particularly in regard to a concave configuration, then the extra expense in providing normal joints might be money well spent.

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[65] "Improved Type of Multiple-Arch Dam," by Fred A. Noetzli, *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 342.

Similar problems arise with concrete face slabs on embankment dams, particularly of the rock-fill type, due to relatively steep slopes and the yield under full water pressure. Diagonal pull at the abutments should be anticipated and provided for.

In contradistinction to most concrete structures, face slabs and reservoir linings of large expanse have been built without joints. In Southern California 0.5% of reinforcing steel in two principal directions has been used with much success. This prevents the formation of continuous cracks and results in the so-called turtle shell pattern of a multitude of short hair cracks.

As to radial joints in arch dams, it is believed that slots on radial planes should be used unless there are very good reasons to the contrary. With the sections next to the slots fully cooled, and a careful job of placing concrete of low water-cement ratio in the slots, there should be need neither for keys nor for subsequent grouting.

Gravity dams, and particularly straight ones, are believed to offer a different problem. They are above all subject to sliding. This is enhanced through uplift and particularly earthquake forces. No place on earth is definitely free from earthquakes. Even if they should not originate in the immediate vicinity of the dam, they could still seriously affect it at a considerable distance. They could and probably would also affect individual blocks of such a dam. Hence, unless a straight gravity dam is designed to resist earthquake forces when under the influence of uplift, keys should be provided for the purpose of mutual support and continuity between the blocks.

The writer has not much faith in the permanence of grout in joints, nor in the permanence of metal and particularly copper seals, unless the latter are well protected and both grout and metal seals are not subjected to continuous movement. Well-designed and constructed keys or slots along vertical joints and the careful treatment of cold horizontal concrete surfaces as outlined by Mr. Blanks[45] in his six general rules are believed to be superior, particularly as to permanence. Proper treatment of surfaces between lifts was often neglected in the past. This was partly due to lack of such knowledge of the characteristics of concrete which has since been acquired through experience and research.

Generally it is believed that carefully designed joints with chamfered edges on the surface and fillets in reentrant angles, preferably with some reinforcing next to the surface, are likely to save maintenance expense and therefore justify the extra initial investment.

It is also believed that the purpose of attaining safe and permanent mass concrete structures is, aside from carefully designed and treated joints, served by using a concrete of moderate ultimate strength containing about one barrel of low-heat cement of proven quality per cubic yard and by placing it with as little water and as much care as practicable. Such a concrete with little chemical heat and excess moisture will be a homogeneous mass of uniform strength; it will satisfy the design requirements of equilibrium of stress, and the factor of safety of the structure it forms can therefore be established more definitely than with a concrete of much higher strength which lacks these essential qualities.

W. A. Perkins,[66] M. Am. Soc. C. E. (by letter).—The design of construction joints is a matter that requires far more attention from designing and construction engineers than it has received in the past, and the profession is much indebted to Mr. Steele for his timely and valuable discussion of this important subject. It can be safely said that, until recently, construction joints, like Topsy in *Uncle Tom's Cabin*, "just grew."

Merely outlining a mass of material which, by its weight or strength, will theoretically withstand the forces developed by the assumed loads does not by any means satisfy the requirements for adequate design. Many other forces enter, such as temperature, abutment and foundation yield, and differences in elastic yield in adjacent sections, which may greatly impair the strength or even wreck the structure. The best design is that which uses the minimum of material so arranged that it can readily yield to the extraneous forces such as those mentioned without losing its ability to withstand the loads which it was built to support. The proper location and design of construction joints plays a very important part in satisfying the foregoing criterion.

The writer agrees with Mr. Steele that the spacing and location of joints should be made to conform as closely as possible with major breaks in the profile of the site after the excavation has been completed. Examination of dams where this rule has not been followed will disclose, in many if not most cases, a diagonal crack extending into the body of the dam where there is an abrupt change in profile. Several factors combine to cause this condition, among them being the differential elastic compression of the concrete and the adjacent rock of the abutment as the load of the upper portion of the dam is imposed and local stresses occur due to difference in elastic yield when water load comes upon the dam, whether arch or gravity in type.

Seldom will the location of outlet works and other features be interfered with by the insertion of an irregularly spaced transverse joint to satisfy an irregular condition in the profile of the dam, as this will always occur either at the foot of an abutment or well up above the bottom. The slight additional expense for form work would be a very small percentage of the cost of the dam and is scarcely to be considered.

Experience would indicate that it is advisable to begin all radial contraction joints at the foundation. Where some of the joints are placed only in the upper portion of the dam, uncontrolled cracks are likely to develop and extend downward, usually in a diagonal direction from the lower end of the constructed joint, a condition noted by the writer in several dams.

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Architectural considerations need not be a bar to selecting joint spacing that will conform to conditions at the site as attested to by the fine appearance of the downstream face of O'Shaughnessy Dam in which there was variation at the abutments from the regular joint spacing.

Discussion of the joints in O'Shaughnessy Dam has been invited by Mr. Steele, and as the writer is somewhat familiar with that structure, a few notes thereon are inserted here. A brief description of the dam follows. Elevations are: Lowest foundation, 3,382; stream bed, 3,500; crest of first stage of construction, 3,726; and crest of completed dam, 3,812. The dam is curved in plan to a radius of 700 ft at the upstream face, and is full gravity type in cross section. The crest length of the original dam was 605 ft and of the completed dam 900 ft.

Joints in the original dam were placed radially with a spacing at the upstream face of 97 ft. When the dam was raised to its final height, these contraction joints were continued to the downstream face, and midway between them an additional joint was placed. At the ends, especially the right, configuration of the abutments after excavation required a different spacing. As stated previously, however, this does not mar the excellent appearance of the downstream face.

Between the old and the new concrete a chamber of an average width of about 5 ft was formed extending from the base of the new work to the crest of the old dam. The inclined weight of the new concrete was carried to the old on ribs 2 ft wide adjacent to the contraction joints and poured monolithic with the new work. Horizontal drains were placed at the top of each step of the old dam, and all surfaces of the old concrete were thoroughly roughened. Likewise, the face of the new concrete adjoining the slot was so formed that, when the slot was filled with concrete, the maximum resistance to movement due to shear, both of the cantilever and of the arch, was provided.

After the new concrete had been cooled to about mean annual temperature, 50° ±, the slots were filled and cooled, and the remaining concrete from El. 3,726 to 3,812 was poured for the full width of the dam, no longitudinal joint being provided above the top of the slot at the crest of the old dam.

Settlement of the new concrete between El. 3,500 and El. 3,726, due to cooling and elastic yield as additional concrete was added, could readily be observed by noting the development of cracks in the supporting ribs, these cracks in practically all cases originating at the outer corners of the old steps.

At the upstream edge of all contraction joints, and also along the crest of the old dam, U-shaped copper water stops were provided. Likewise, grout stops were provided whenever necessary in all the new concrete, including horizontal stops at 50-ft vertical intervals. However, to provide for grouting all joints, both old and new, it was necessary to calk all joints in the old dam, except where water stops were used, and also in many places where the old concrete adjoined the new. As there were no horizontal stops in the old dam, it was necessary to exercise great care in bringing these joints under hydrostatic pressure before inserting grout to avoid side deflections of the blocks, thus allowing the calking in the joints that opened to loosen. Aside from a little trouble because of this loosened calking, no difficulties were encountered in the grouting operations.

Numerous inspection wells and galleries are provided in this dam, including inclined galleries along all the contraction joints of the old dam at the contact between the new and old concrete, from El. 3,500 to El. 3,730.

An inspection of the dam was made by the writer soon after the completion of the new work with water within 4 ft of expected maximum storage level. The total observable leakage at that time would not exceed 50 or 60 gal per min, and the major portion of this came from step drains near the right end and was believed to enter through a crevice in the rock of the abutment which was not entirely sealed by the grouting operations. Another inspection made recently, with water surface about 30 ft lower than at the first inspection, revealed a leakage about half as great.

Observation of the supporting ribs adjacent to the inspection galleries and readings of strain meters so far indicate no differential movement between the new and old concrete or any other action contrary to expected behavior. It is fortunate that in this dam the "sun face" is on the downstream side where the greater movement due to the wider range of temperature change is provided for by the additional joints placed in the new work. Since completion of the work, in 1938, no cracks have been discovered so far as the writer is aware.

The writer fully agrees with Mr. Steele in his statement concerning longitudinal joints. This is a matter which has not received sufficient consideration in the past, but with the increase in the size of dams it should be given more attention. Cracks develop sometimes in the most unexpected places, and a longitudinal crack in a large gravity dam may be so located as to seriously impair its stability. Furthermore, the probability of discovering an internal longitudinal crack is much less than observing the development of a transverse crack. This is a matter which merits the serious joint consideration of designing and construction engineers.

Mr. Steele questions the advisability of the use of keys in joints. It is true that this may be a heritage from the past, just as the early paved roads inherited their high crowns from the old dirt roads. However, this is not sufficient justification for scrapping the keys entirely. In the case of arch dams engineers need to have assurance that the sections of the dam will lock together securely to produce the proper arch action before cantilever action due to rapidly rising water against the higher sections has produced dangerous vertical beam stresses.

The use of keys and their type is a matter of design fully as much as any other part of the structure. In considering this feature, certain questions are pertinent. Will the joint be weakened by the use of keys? Does the proposed type best serve the purpose for which it is intended? If keys are omitted, how will the structure behave when placed under load? In arch dams having heavy overhang at the crown, a proper system of keys is necessary in order to give stability to the structure through arch action without the development of undue cantilever stresses until such time as the dam can be properly grouted. The use of slots in this type of dam is not practicable.

Use of slots instead of contraction joints for the closure of a dam does not necessarily guarantee that transverse cracks will not develop at some subsequent time. In this respect, the use of joints with adequate water stops at the upstream face better permits the movements to take place at controlled points, eliminating the formation of uncontrolled cracks.

As the weakest link determines the strength of an entire chain, so do joints between successive pours of concrete determine the strength of a dam or other structure. The wide divergence of opinion among engineers as to the best method of cleaning cold joints indicates the interest that constructors are taking in this feature of the work.

The cleanup really begins when a pour is being completed. It is highly important that the moisture content of concrete be sufficiently low to prevent the accumulation of surface water and flotation of the lighter particles of the cement. Vibration should be continued a sufficient length of time to consolidate the mass, and surface working should be limited to that necessary to eliminate gravel pockets and loose pieces of aggregate. This should be done by forcing the pieces of aggregate into the mass by booting or with tampers rather than bringing fine material to the surface by puddling. After pouring, the surface of the concrete should be protected from all disturbances, after initial set has begun, until it is sufficiently hard not to be injured by traffic.

Specifications should be so written that the construction engineer will have sufficient latitude to select that method of cleanup which will give the best results, and he should not leave this important item of the work to the care of an inexperienced inspector. With proper manipulation, performed at the right time, good surfaces can be secured by any of the various means that have been used. On the other hand, the best method yet devised will not be satisfactory if slipshod methods of operation are used.

In the opinion of the writer, the mortar that is spread on a cold joint ahead of a concrete pour should be of practically the same mix as the mortar of the concrete that is being used. It would be interesting to do experimental work with gunite on a cold joint to study this type of treatment. Instead of using a layer of mortar it is suggested that a thin layer of rich gunite, using fine sand, be spread on the hard joint surface just ahead of the advancing face of the new concrete. The cost should be no greater than the use of mortar, and the results might be more satisfactory.

In gravity dams, where sliding is important, the factor of safety is increased by sloping the joint surface slightly upward toward the downstream face. This increase amounts approximately to twice the tangent of the angle of the slope.

There is still room for much improvement in the design of satisfactory water stops at construction joints. There is serious objection to the type commonly used in the United States—namely, a copper sheet bent in a U-shape with two long wings buried in the concrete. If a leak develops at a brazed joint or by a piece of sharp aggregate piercing the plate during movements of the dam, repair is impossible, and the value of the entire water stop is lost. Also, unless the stop is buried deeply, there is danger of the concrete outside the embedded wing breaking off. In the case of a thin arch dam, although it is designed on the assumption of the arch loads being carried by the full thickness of the concrete, yet at each vertical construction joint a large percentage of this thickness is lost because of the necessary depth at which the water stop and, on the downstream face, the grout stop are buried.

A bent copper plate with an exterior protective plate held in place across the joint by liner bars and anchor bolts is one substitute that could be used. A design embodying this idea was used at the Bissorte Dam, shown in Fig. 13 of Mr. Steele's paper. It is believed that a design could be developed, however, that would be somewhat less expensive than that illustrated in Fig. 13.

Construction joints are an important feature of all concrete structures, especially dams, and merit the most careful consideration of designers and specification writers. Their work, however, is completely nullified without the intelligent "heads-up" cooperation of a competent and alert construction organization.

L. J. Mensch,[67] M. Am. Soc. C. E. (by letter).—High dams exert enormous pressures and shears on the foundations and abutments. Nearly every contributor to this Symposium has laid particular stress on the importance of the thorough exploration of rock conditions surrounding the dam so that proper grouting procedure may be planned in order to make the foundations as nearly as possible "bedrock."

This perfect agreement between experts leads one to the conclusion that serious troubles must have been encountered with some high dams after they had been built. Messrs. Houk and Keener refer to cracks, seams, fissures, joints, and bedding planes in all masses of rocks, in view of which the tests on cubes of sample rock which are generally very carefully faced on all six sides will hardly give a proper answer to the question of the actual strength of rock, in place.

Ultimate strengths of test cubes vary from 3,000 to 45,000 lb per sq in. in compression when tested dry. Many rocks absorb as much as 10% of water, and test cubes, when tested wet or with oiled surfaces, will show strengths of only a fraction of those tested dry. Shearing strengths of rocks are very low —as low as 150 lb per sq in. in some granites and probably not more than 1,000 lb per sq in. in the best granite ever tested.

Values of safe shearing stresses of 300 to 700 lb per sq in. as mentioned by Messrs. Houk and Keener are astoundingly high, and when considering the dip of the bedding planes of some granites or similar rocks, the only safe assumption is to consider the base as so much sand, when the shear-friction factor of 0.65 will appear rather high than low.

A high dam like Boulder Dam, if it acts like a gravity dam, exerts a horizontal shear of 8,000 tons for each linear foot of the longitudinal axis of the dam. How is this great force transmitted to the shattered and partly grouted masses of rock beneath, downstream, and sidewise? One can imagine secondary horizontal or inclined arches acting in the rock which are affected by the horizontal shearing forces and by the inclined compressions from the downstream side of the dam. The rock may be weaker than the concrete, especially downstream and to the side of the canyon where no grouting is done, and the secondary arches will be very much wider than the base of the dam. If such arches are forming, then one can see the necessity for extending the grouting operations far beyond the base of the dam.

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There is no doubt that any dam, whether a gravity or arched structure, with proper abutments will act like a curved plate; but this plate has cracks and joints in three directions and the only safe way is to judge the structure as if built of jointed brickwork. Tests on reinforced brick beams have shown[68] that such jointed work fails when the shear reaches a value of about 40 lb per sq in. The 8,000 tons acting in Boulder Dam, provided it were a gravity structure, on a surface 1 ft wide and 650 ft long (the width of the base), mean that the shear would be 171 lb per sq in., a value probably four times as high as a jointed structure could sustain. Hence, it is evident that Boulder Dam acts as a curved plate and not as a gravity structure, notwithstanding that the base is nine tenths of the height.

However, assuming that this curved plate were an entire monolith, there are some very serious movements of which Messrs. Houk and Keener did not make any mention. Assume near the bottom of the dam a prism or prismoid cut out of the dam, 1 ft square and 650 ft long, the 1-ft end to be at the upstream face of the dam, and the slightly smaller end at the downstream face. The water pressure acting at the base of the dam is 45,000 lb per sq ft, which will act on the upstream face of the prism and this pressure will decrease to zero at the downstream end. Lamé's theory of thick pipes acted upon by outside pressures will afford an idea as to how the compressions on the cross sections of this filament will diminish with the distance from the upstream face. The compression will decrease more slowly than according to a straight-line variation from a maximum at the water side to, say, zero at the air side. Supposing that the average compression is only $\frac{1}{2} \times 45{,}000 = 22{,}500$ lb per sq ft, the shortening of this prismoid will be given approximately by $\frac{22{,}500}{144} \times \frac{650 \times 12}{2 \times 10^6}$ $= 0.61$ in.; this is the relative movement between the two faces of the dam.

Unless the underlying rock has the same movement between the two faces of the dam, a shear crack must open either in the concrete above the rock or in the rock beneath the concrete. The downstream face of the dam has its own deflection, however, whether it acts as a horizontal arch or a horizontal beam, which will still further induce relative movements between dam and foundations. Conditions are certainly serious at the contact of dam and foundations in a dam of the Boulder type. In a dam only 360 ft high, the movement will be only one fourth as large.

Tangential shear, twisting moments, and beam actions are effects which are very little understood even by high-class stress analysts. These actions have been considered by A. E. H. Love[69] for a thin elastic plate. A practical analysis by H. Leitz,[70] based on Professor Love's theory on the design of rectangular plates supported on four sides, led to erroneous results which differ from test results by several hundred per cent. Professor Love shows how the twisting moment may disappear at any point of the curved plate by finding the section in which the principal stresses are acting. How this can be done in a simple way has been demonstrated by Prof. Arturo Danusso[71] for both rectangular and triangular plates. Professor Danusso's formulas agree exceedingly well with actual tests to destruction.

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[68] *Transactions*, Am. Soc. C. E., Vol. 99 (1934), p. 755.

[69] "The Mathematical Theory of Elasticity," by A. E. H. Love, Cambridge, 1906.

[70] "Forscherarbeiten auf dem Gebiete des Eisenbetons," William Ernst & Sohn, Berlin, Heft 23, 1914.

Where the canyon is of a rectangular shape, the system of horizontal arches and vertical cantilevers is the best approach for a proper stress analysis; but where the canyon approaches a triangular shape, the arches in the lower part of the dam should be assumed inclined, on the lines indicated by the writer,[72] and the cantilevers should be assumed normal to the foundations and abutments.

As far as the writer can see it will take many years yet before the stress problem of the high dam will be solved to the satisfaction of conservative engineers, and agencies responsible for the design of large dams should give serious consideration to types of dams other than the solid gravity or solid arched dam, especially where the foundation conditions may not be ideal.

The paper on concrete control in this Symposium is a fine exposition of the present stage of concrete practice in dam construction.

The paper on construction joints by Mr. Steele is highly interesting and shows the perplexing nature of joints and cracks. Mention is made of cracks forming during concreting and very soon afterward. They are the effect of volume changes due to chemical actions, moisture, and temperature variations and very often, on hot days, to the sun rays beating down on the fresh concrete or on the forms. Much less cracking occurs in winter than in summer work.

Joints and cracks cannot be prevented in dams, and the designer must take this fact into consideration. As stated by Mr. Steele, keyed joints in concrete dams and retaining walls (and tanks) are a hang-over from the mortise and tenon used in timber construction, and the fourth paragraph under "Plain Versus Keyed Joints" deserves to have been printed in bold face type. This detail is one of the worst features in common design practice. Such keys are expensive to build; the depressions form sinkholes for the accumulation of dirt, wood shavings, etc.; they are difficult or impossible to clean out; and cracks are forming either from shrinkage or the removal of forms. The tests by M. Considère[73] on closed joints with steel dowels have demonstrated that such joints fail at a unit stress in the dowels of from 30,000 to 40,000 lb per sq in. Dowels form an inexpensive means for making joint connections, but no such mention has been made in any paper of the Symposium; therefore, one must conclude that designers have not yet used dowels to any extent in joints of dams.

Water stops of metal are another perplexing problem. Such stops have not proved successful in road construction and have been abandoned by most highway departments after a trial for several years. Neither have they been successful in high dams. No tight contact between the flanges of the seal and the concrete can be counted upon, even where vibration is used. Tiny air bulbs seem to stick to the metal and prevent contact, or in case of vibration, air will accumulate at the flanges. Movements of the adjoining concrete surfaces have caused cracks in the crimped portion of the seal, thereby making it worthless as a water stop. The flanges cause cracking of the surrounding concrete, and the wider the flanges the more serious the cracking will be.

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[71] *Il Cemento* (1911) (first 10 issues).

[72] *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1262.

[73] "Expériences, rapports et propositions instructions ministérielles relatives à l'emploi du béton armé," Commission du ciment armé, Paris, Dunod, 1907.

Asphaltic joint fillers have not proved a success in most joints; where there are large movements the filler has been squeezed out to such an extent that the bare concrete surfaces have been found in contact. Boards or planks of wood with protective treatment make a cheaper and better filler; under a pressure of 1,000 lb per sq in. across the grain they will compress to one half their thickness without extrusion.

LEWIS H. TUTHILL,[74] ESQ. (by letter).—In his comprehensive paper and references Mr. Tyler has given an excellent résumé of the fundamentals of good concrete making for masonry dams. There are two aspects of this subject, however, which may well receive brief amplification in discussion.

The first point is emphasis on the importance of uniformity of aggregate as the very foundation of concrete control. It is true that concrete in place may be greatly benefited by use of vibrators, special cements, cooling systems and absorptive form lining regardless of the uniformity of the aggregate as batched; but these highly important elements do not constitute the control which is responsible for the efficiency and the uniformity of fresh concrete as a manufactured product for use as a construction material. The most efficient concrete is that having the minimum cement and water contents compatible with adequate workability, sufficient strength, durability, and other essential properties. Unless aggregate is uniform in grading, cleanness, and in silt and undersize content, an appreciable extra margin of cement (and possibly of water also) must be provided to insure quality and workability under all conditions of aggregate variation. Uniformity needs no definition, and its importance as a property of concrete should be obvious.

The now recognized practice of using five or six divisions of aggregate for mass concrete is a long step toward aggregate uniformity as batched when compared with the fairly recent use of two or three sizes with the one division of coarse aggregate regarded by some as a needless refinement. Screening equipment has become more efficient, and current specifications are usually more particular about the efficiency of screening and percentages of undersize. On larger work, however, the necessity for an ample standby supply, and for more frequent rehandling of the material between aggregate plant and batchers, often results in a variable undersize problem which, at times, may largely nullify most of the efforts made to produce uniform concrete. Variable quantities of fine gravel from among each of the sizes are principally responsible for this difficulty.

The percentage of pea gravel (No. 8 to $\frac{3}{8}$ in.) in a concrete mix is the most critical of the several aggregate sizes, particularly in mass concrete, because an efficient but amply workable mass mix contains less mortar than corresponding mixes of smaller aggregate. With lower mortar content there is consequently a smaller surplus available to overcome changes in the aggregate which demand more mortar if workability is to be sustained. The percentage of pea gravel is critical because more mortar is required to maintain workability when a unit of pea gravel is added than for a similar unit of any other size of coarse aggregate. On account of breakage and imperfect screening each bin of a coarse aggregate size is a source of pea gravel that varies in percentage depending on how the undersize aggregate may accumulate and vary in its flow to the gates (see Fig. 24). Material coarser than No. 8 in the sand is another source, and it may vary in percentage with changes in grading of the source material. Since variations in the quantity of pea gravel entering the batch may vary as much as 100% from that desired, due to certain unfavorable combinations of situations in the various aggregate bins, it is clear that if uniformity is to be obtained means must be established for keeping the smaller fractions of undersize at a minimum and flowing uniformly into the batchers batch after batch. (Undersize in larger aggregate that is but little smaller than those sizes is usually not significant since equivalent proportioning changes in the mix seldom produce a noticeable difference.)

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[74] U. S. Bureau of Reclamation, Denver, Colo.

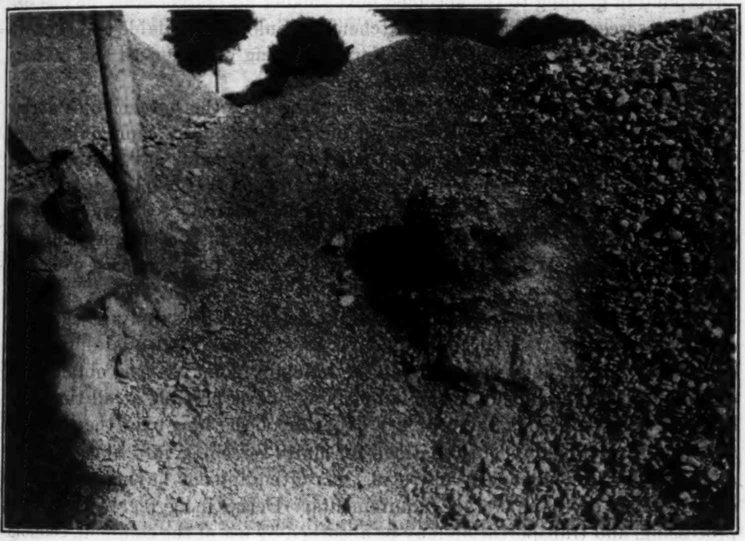

FIG. 24.—CONCENTRATION OF UNDERSIZE AGGREGATE AT POINT OF DISCHARGE INTO STOCK PILE

Excessive percentages of pea gravel sizes in sand due to the grading of the deposit can be removed by proper screening processes, and that remaining in wet sand will give little or no trouble due to segregation in the bins. In dry sand, however, any appreciable percentage of material larger than No. 8 will segregate in handling and in the batcher bins, and will give very serious trouble. The most effective correction in this case is to use a $\frac{1}{8}$-in. screen for sand and to put the pea gravel (screened through a ½-in. or ⅝-in. screen) in a separate bin. In fact, much better control is afforded with either wet or dry material by keeping pea gravel of these sizes separate where aggregate in the mix is graded to a maximum size larger than 1 in.

The solution of the difficulty from undersize (after due provision has been made to keep breakage minimum for the sake of preserving the usually deficient supply of large sizes) is finish-screening at the batcher bins. When this is done all previous screening can be very rough, and the usual overlapping and segregation in economical conical stock piles over conveyer tunnels can be ignored. By adjustments in simultaneous feeding of all sizes on to the belt from the stock piles supplying the finish screen, the desired balance of finished materials in the batcher bins can be maintained. Since any undersize or oversize will not correspond in grading to the specified next smaller and larger sizes, no one size should be screened singly over the batcher bins unless dependable and effective means are provided to insure the uniform blending of undersize and oversize with the next smaller and larger sizes.

By following the practice of finish-screening over the batcher bins during construction of the Colorado River aqueduct, the difficulties commonly resulting from stock-piling were largely avoided to the benefit and satisfaction of both construction and engineering interests, and the practicability of such procedure was demonstrated. Operations requiring materials for as much as 100 cu yd of concrete per hour were efficiently supplied in this manner and with surprisingly few delays, although only one screening unit was used over each plant.

It is recognized that there must be no delay in the supply of aggregate after concreting starts, especially on a very large dam. Adequate protection from delay due to finish-screening at the batching plant may be obtained (1) by maintaining the bins over the batchers as full as possible at all times; (2) by somewhat increasing the ordinary capacity of the bins over the batchers; and (3) by dividing the installations for finish-screening into two, three, or more parallel, independently operating sets of equipment having a combined output well in excess of average requirements. Delays in aggregate excavation, processing, and transportation may be absorbed by use of the common, economical conical stock piles over a conveyer tunnel near the batching plant. Since handling of finished material regularly through this ordinary type of large-volume storage develops objectionable variable segregation of the character shown in the illustrations, and standby ground storage of finished aggregate (unused except in an emergency or at the end of the job) involves expensive handling operations, the provisions suggested for avoiding serious delay in connection with the proposed finish-screening are relatively inexpensive in view of the superior uniformity of aggregate obtained. It is recognized that in many cases it may not be economically feasible to change over established job setups in order to do the finish-screening at the batching plant. For best economy and results the aggregate processing, handling, and batching plant must be arranged at the outset for this plan of operation. Perhaps it should have its beginning as a specification requirement.

For structural economy in the batching plant for the very large jobs, it may be necessary to locate the finish-screening equipment adjacent to, rather than above, the batching plant. Conveyers or elevators for each aggregate size should then be provided.

Except where materials are dry, the sand should not be finish-screened with the coarse aggregate but should be separated from it and washed, and processed if necessary, at the aggregate plant. This leaves three alternates for disposition of undersize in the first size of gravel larger than sand: (1) It may be wasted; (2) it may be arranged to blend uniformly with the sand when the latter is being run into the sand bin (provided it does not impair the grading of the sand); or (3) the range of limiting dimensions of the fine gravel may be reduced (to ½ in. or ⅝ in. maximum), and as a result its percentage in the mix may be limited, such that variations in its percentage of undersize will be unimportant, making a finish-screening on its minimum size comparatively unnecessary.

When finish-screening is performed at the batching plant, screening at the aggregate plant may be considerably simplified. Aside from separating the sand (if it is a wet plant), scalping oversize, and wasting excess in certain sizes (often a portion of the sand and pea gravel), little screening of coarse aggregate need be done except perhaps roughly into two or three sizes in order to expedite later adjustment of the feed from the stock piles to the finish-screens so that material will maintain a balance in the batcher bins. Whether roughly separated or not, if the coarse aggregate may be drawn through a number of adjustable gates under the storage piles, the operator will usually be able to maintain a good balance among the aggregate sizes.

The other point which might be mentioned is the remaining (though narrowing) lag between what is known and should be done about concrete control, and what actually is done about it on the job. Under the heading, "Conclusion," Mr. Tyler says: "Efforts toward * * * control * * * are becoming more successful * * *." It is not clear whether he means that this is because control methods in themselves are improving and are better applied, or that control methods as such are being adopted more readily by management. It does not greatly matter which, since either means a better net result, but why should there be, or have been any serious question on the part of management as to the desirability of practicable measures of control? Too often progress has been impeded and opportunities for obtaining superior results have been lost by an attitude of management toward control indicated by such remarks as "If we let these cranks have their way they will ruin the job." If this were only the attitude of the occasional superintendent it would be no cause for comment, but to a various degree it is yet to be found among engineers, and they, of all people, should be aware by this time of the true meaning and value of concrete control, and should lend their considerable influence toward an invariable application of its simple principles. When management recognizes that concrete control is merely the application of all practicable means of getting the most out of the concrete construction dollar, the costly lag between what is known and what we do will be appreciably shortened. Mr. Tyler may then say, "Efforts toward control will be successful."

F. A. Nickell,[75] Esq. (by letter).—Even in application to engineering structures geology is a broad field. Mr. Crosby has done a remarkably thorough job in outlining the more common geological problems in the selection and investigation of dam sites. As a general thing different types of rocks have certain outstanding characteristics so that principles can be defined and applied to sites where these rocks occur. The subject does not lend itself readily to simple classification of conditions, nor to formal treatment, but examples are of considerable value since they reflect experience.

Each dam site presents problems in a way different from those previously encountered. General concepts about foundations are difficult to apply in solution of specific cases, and there is danger in drawing a close analogy between sites having apparently identical geology. In his introductory statement, Mr. Crosby points out that the success of a dam depends on many factors. It is necessary to know the significance of features in the foundation as a basis for correct design. Although the nature of formations is important, the critical response in rocks due to influences coming from the dam is commonly the effect of small adjustments within members. This merely emphasizes that minor features, such as seams, may determine the ultimate success or failure of the structure.

The weight of the dam and thrust from reservoir storage creates conditions of concentrated stress that the rocks have not experienced before, perhaps even prior to erosion of the canyon. The resulting deformation suffered by the foundation is in accordance with the net physical character of the component rocks, and thus may be largely elastic or purely a matter of consolidation. Although the extent of probable adjustment can be computed with reasonable accuracy from a knowledge of materials in the foundation and is, of course, extremely important to the designer of dams, continued submergence may cause a decrease in available support through combined effects of saturation and removal of soluble components. Mr. Crosby's point in regard to employment of an experienced geologist during investigation and early construction is, therefore, well considered.

There are two paramount objectives in all construction: First, to build permanently, and second, to obtain efficient operation. It should be recognized that these objectives are modified according to local conditions and purpose of each project, but security of the dam should never be in doubt. What constitutes efficient operation is subject, as Mr. Crosby indicates, to the location and general requirements for storage. A leaky dam site may permit adequate regulation of floods, provided seepage is noninjurious, but a similar loss would be unduly costly to an irrigation project in an arid region. Furthermore, the significance of certain geological conditions may be widely different at the dam site and in the reservoir area. Reservoir seepage in certain cases is permissible, but the same amount of leakage might be dangerous around the dam where the percolation path is short.

The stability of foundations is contingent upon bearing power, shearing strength, and resistance to sliding, under the assumption that the concerned rocks retain their original character—in short, will not soften, be dissolved, or lose constituents by piping. These factors can be computed in the laboratory and, provided the samples have been carefully selected, the values so determined for compression, shearing strength, and frictional resistance are ordinarily sufficient for design of most dams. How representative the measurements are depends on whether unknown defective or weaker members exist in the foundation. It follows, therefore, that exploration of the dam site should be so exhaustive that no reasonable chance exists of overlooking critical features that may cause failure. This points again to the need for the expert knowledge of the geologist, and, moreover, one familiar with engineering.

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Geological Investigation.—The value of exploratory results depends alike on thoroughness of investigation and accuracy of interpretation. Mr. Crosby has laid considerable emphasis on the need for careful geological appraisal so that any discussion of exploratory methods based on another's experience involves some repetition.

Dam sites are chosen by aerial, topographic, and geological comparison. Investigation is a matter of securing adequate information for design of all suitable types of dams. The diamond drill is used extensively in both preliminary and final exploration. It is desirable to have a hole at least 2 in. in diameter for maximum core return. Recovery of representative material alone is not enough. Usually percolation tests with packers or casing are required to segregate leaky horizons where grouting or other protection is needed. In preliminary studies the configuration of the water table is important since it reflects the course of natural ground-water movement. The elevation of the water table is determined by recession to a static level or ascension following bailing. There is a trend toward large borings with shot drills allowing periscopic and visual inspection of the hole. Few sites for high dams are explored satisfactorily without tunnels, shafts, and trenches at several levels.

The interconnection of open cracks through the rock may have great importance. Leaks discovered by packers and other devices can be traced with coloring fluids, diagnostic liquids, or electrolytes. Some unusual methods have been devised by enterprising investigators. Free passage to the river along openings in lava at a proposed site in Oregon was shown by appearance in the stream of kerosene dumped into a neighboring drill hole. The spread of joints and solution channels at the Alcova Dam site, in Wyoming, was vividly indicated during investigation by a burst of compressed air into an angle hole passing under the river.

Mr. Crosby has mentioned the use of geophysical methods in dam investigation. The possibilities are not generally appreciated and the interest of geophysicists should be aroused in engineering and foundation problems. The identification of porous media by the electrical resistivity method has not been utilized in analogy of seepage through permeable foundations. The interpretation of seismograph records, based on elastic properties of rocks, might disclose a relation between seismic response and deflection under high dams. The period of foundations and resonance of structures during earthquakes are matters of vital concern in seismically active areas and might be established in the same manner.

The percolation path in permeable layers is closely allied to the design of the so-called "floating" structures and with uplift under other types of dams. In many instances the slope can be portrayed by drawdown measured in small holes around a large central well used for pumping.

The writer has been impressed by the desirability of having field tests to check experimental values determined in the laboratory and to eliminate the element of personal judgment so far as possible. Experiments made with ease in the laboratory can be conducted only with difficulty, if at all, on rock in place; yet the added strength due to confinement affords an undetermined factor of safety that might be utilized for closer design if conditions could be positively analyzed. Definite progress in this direction is seen on the part of many construction agencies. The compressive strength of sediments at the Table Mountain site, in northern California, and of granite at the Twin Springs site, in Idaho, has been calibrated by loaded column and hydraulic jack with rather uniform results. An elaborate program of bearing tests is being conducted along zones of weakness as well as in normal rock during excavation and preparation of the foundation for Shasta Dam, in California.

Foundations of Typical Rocks.—The trend in design is an expression of accumulated experience with earlier dams. Because the response of materials in concrete dams can be governed fairly well by strict specifications, foundations remain the most questionable element in design. A reasonable effort in exploration and instrumental measurement will repay in ultimate benefits many times its initial cost.

Naturally, the problems vary according to the composition of foundations. In the introductory remarks the writer indicated that generalization is usually unwarranted, but that examples ordinarily are instructive. Mr. Crosby has maintained this idea in presenting illustrations of dams on different types of foundation materials. Most large concrete dams—for example, those built by the Bureau of Reclamation—are constructed on foundations of hard rock. Hard rock may be taken to include granite and other bodies of deep-seated origin, as well as certain sedimentary, metamorphic, and volcanic formations, which, considered as a group, offer foundations generally acceptable for any type of dam. Although hard rocks are considered best suited to high dams, they have common weaknesses, including crushed zones, joints, contacts, irregular weathering effects, and laminations of foliate rocks. The importance of most defects diminishes with depth. An empirical rule holds that excavation should be carried to levels where support is adequate and at which the rock below can be treated satisfactorily—for example, by grouting and drainage.

Dams on Granitic Rock.—Mr. Crosby has described treatment of faults in foundations of granite and the remedy is more or less orthodox for inactive fault zones. Crushed zones in the foundation for Seminoe Dam on the North Platte River, in Wyoming, were excavated of all soft material and refilled with concrete. A less serious feature exposed in the floor at Grand Coulee Dam, in the State of Washington, was excavated some distance below the average level of the adjacent rock. Bartlett Dam on the Verde River, in Arizona, is a multiple arch, 286 ft high, on a foundation chiefly of rather fine-grained granite. Strong joints parallel the canyon, forming a series of vertical rock slabs on the right abutment where buckling might occur without sufficient excavation and adequate lateral support.

Formations in some regions have characteristic defects. The Boise Mountains in Idaho, largely of granite, contain numerous broad zones of minutely sheeted and friable rock. This condition would have a considerable effect on excavation for a dam—for example, at the Twin Springs site on the Boise River.

The common problems in preparation of foundations of granitic rocks are well defined and resolve largely into questions as to depth of excavation and requirements for grouting and drainage.

Dams on Metamorphic Rocks.—Foundations of metamorphic rocks may be entirely acceptable but have an element of added danger in foliate structure. One of the contributing causes in the failure of the St. Francis Dam, in California, as Mr. Crosby indicates, was the schistosity paralleling the left abutment face. Weathering in banded rocks is usually selective so that decay persists irregularly below the average level of unaltered material. Since sliding is the main hazard, added resistance may be given the foundation, as was done at the Easton Dam on the Yakima River, in the State of Washington, by providing a saw-tooth outline.

Dams on Volcanic Rock.—Mr. Crosby's few comments on dams and reservoirs in volcanic rocks barely touch upon the innumerable problems associated with formations of volcanic origin, which, as a group, are unpredictable. Brecciated contacts, contraction joints formed during cooling, lava tunnels, porous intravolcanic sediments, and a depressed water table are ordinary difficult features. Volcanic rocks occur over vast areas, including a large part of the Northwest and future demand for storage in those regions will continue to raise perplexing geological problems. Reasonably effective storage can be obtained where the lava in the reservoir area has a blanket of sediment, as is the case at the American Falls, Minidoka (Idaho), and Wickiup (Oregon) dam sites.

Without protective cover, impervious beds in an alternating series of flows and sediment may limit losses. A secure dam can be built usually with provision only against uplift and piping, if the lava foundation offers adequate support. Various remedial measures have been successful, including grouting, concrete cutoff walls in the permeable layers, and blanketing. It is evident that a satisfactory dam in volcanic rocks may become ineffective and possibly unsafe if the reservoir is raised and storage increased.

Dams on Sedimentary Rocks.—Sedimentary rocks common in every section of the United States are in the foundation of a large number of dams. Conditions of deposition and later modifying effects tend to produce variations in character even among similar kinds, and the illustrations offered by Mr. Crosby can represent only a few of the problems connected with them. Formations of the same geological classification vary widely in physical ability according to texture, bedding, degree of cementation, sequence of strata, and internal structure, so that, irrespective of the geological age of the concerned rocks, the situation at each site has to be analyzed separately.

Sandstone with great frictional resistance internally and between layers is generally well qualified for moderate to heavy loads. Limestone and certain chemical equivalents may be highly suitable on one hand or dangerous otherwise, depending on geological processes experienced by these relatively soluble rocks. With added clay content, complications also increase. The acceptability of these sediments is crudely expressed by a graph with sandstone, limestone, and clay at the apexes of a triangle, representing, respectively, "best," "good," and "fair" quality stone.

In the future, more dams (many of concrete on account of spillway and other requirements) will be built on rather soft foundations. There will be a need at these sites for field investigations on bearing ability such as are described by Mr. Crosby and which have been made in detail at the Conchas Dam in New Mexico and for the Possum Kingdom Dam in Texas.

Dams on Pervious Foundations.—Mr. Crosby neglected to discuss masonry structures of restricted height on pervious foundations. Dams across streams with deep alluvial cover on bedrock are occasionally necessary. Since it is not practical to found them on solid rock, and because spillway requirements are usually exorbitant, the dam is designed as an overflow structure, secure from uplift and sliding, and obtaining support by means of a reinforced-concrete matte of adequate length, in cases resting on a forest of piles. The Imperial Dam on the Colorado River, in Arizona and California, involves every aspect of this kind of problem—a deep, pervious fill and a vigorous stream. The design was made on the basis of laboratory tests of representative material to determine percolation path, sliding resistance, internal friction, and bearing. Geology has little to offer directly in this kind of situation, but should be valuable in analysis of material, studies of retrogression, and development of stream characteristics.

Summary.—A conservative principle in construction is the adaptation of design to the foundation. Because reasons other than geological limitations govern the selection in many cases, the best plan is reduction of unit load to within safe limits. Exhaustion in the number of unused good sites, the extreme conditions introduced by very high dams, and the tendency to build where necessary regardless of the geological complications accentuate the demand for adequate knowledge and ways of obtaining it. The paper by Mr. Crosby gives a broad cross section of common problems connected with different types of foundations and is suggestive of the things to look for during investigation. An opportunity for study is offered by each new dam. Besides thorough field tests to corroborate laboratory experiments, a valuable source of information will be realized in observations subsequent to construction. Since concepts of foundations are speculative as to the long-time effects of load and submergence, continued measurements are desirable during operation following completion of the dam. For this purpose various instruments are available, including pressure cells, stress meters, strain meters, and tiltmeters embedded in rock beneath and downstream from the dam to indicate distribution of stress, bulging or differential adjustment, excessive uplift, and other pertinent data. It is encouraging to note the installation of instruments at some dams in reflection of a growing appreciation of foundation problems.

Leslie W. Stocker,[76] M. Am. Soc. C. E. (by letter).—In setting forth general considerations governing the arrangement and details of construction joints in dams, and in presenting facts, and deductions by himself and others from such facts, as to joint treatment in existing dams and the behavior of some dams after construction, Mr. Steele has rendered a service for which the designer of a dam should be very grateful.

A 5-ft height of lift has been common for many years, doubtless originating with practical construction considerations, and, having been found to give satisfactory results, in general, has become practically standard. The spacing of transverse joints has been decreasing from the time, not a great many years ago, when they were first introduced, to the distance of about 50 ft which now prevails. Data in Table 11 indicate that approximately 10 to 20 years ago a number of designers thought favorably of the practice of using a very wide spacing in the lower part of a gravity dam, halving it above, and in some cases quartering it at a still higher level. This appears rather venturesome, unless a long time elapses between the times of pouring the concrete placed above and below the elevations at which the spacing changes, as there must be considerable tendency for the formation of a crack as a downward extension of the partial joint. It would be interesting to know the reasoning behind the use, in the upper part of the Cignana Dam, of the partial joint extending only part way from the upstream face; this seems a direct invitation for a crack to form as a continuation of the joint through the downstream concrete.

In the case of the enlarged O'Shaughnessy Dam of the City of San Francisco (dam No. 60 in Table 11), the construction joints (see Fig. 25) had to be so treated as to perform the duty (unusual in high dam construction) of transmitting stresses from the original section into a large addition in such a way that on completion the addition should bear its proper share of the load, the work of enlargement being done while the original dam was in service under varying load conditions. So far as that function is concerned, the principal construction joint is the longitudinal one between the old and the new work. The design of the joint along the original downstream face is of particular interest.

O'Shaughnessy Dam is of the curved gravity type, having a radius of 700 ft measured to the upper (vertical) part of the upstream face. Its height is 312 ft above stream bed, or 430 ft above the lowest point in the foundation. It was originally planned for its present height, but on account of economic considerations was initially built only to 226.5 ft above stream bed, leaving 85.5 ft for future addition. This initial work was completed in 1923. The base section, below stream-bed level, was built to the full thickness required for the ultimate structure, to avoid future cofferdam work. A block containing six outlet valves and their discharge conduits was built to full thickness for its entire height. The remainder of the original dam had only the thickness necessary to suit its own height, leaving a level concrete surface 80 ft wide at the stream-bed elevation to serve as foundation for the future addition. The downstream face was formed in 5-ft steps corresponding to the lifts of concrete. The radial construction joints of the original work are spaced, in general, 97 ft apart at the upstream face. The south end block, 130 ft long, cracked transversely 85 ft south of the southerly joint; nothing indicates that the spacing is too great elsewhere. The radial joints were not grouted. Sheet copper water stops of the U-type were placed across these joints near the upstream face; a longitudinal copper water stop was provided along the crest, with its lower half embedded and the upper half folded down and enclosed in a wood-covered recess, to be unfolded and become embedded in the new concrete of the later addition. A water stop similarly embedded and housed was set around the perimeter of each of the 18 siphon spillway ports.

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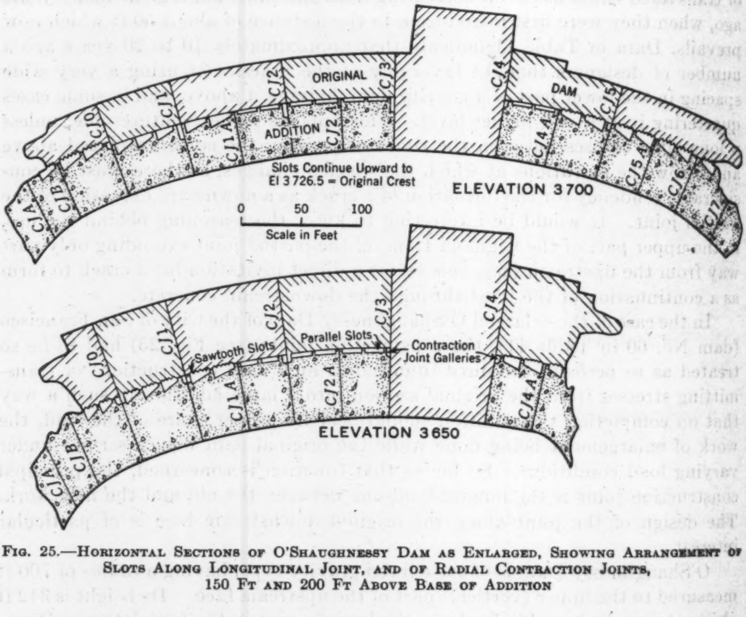
CJA
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ORIGINAL
ADDITION
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CJ14A
DAM
CJ15
CJ15A
CJ16
CJ17
Slots Continue Upward to
El 3726.5 = Original Crest
ELEVATION 3700
0 50 100
Scale in Feet
CJA
CJB
CJ10
CJ11
CJ11A
CJ12
CJ12A
CJ13
CJ14
CJ14A
Sawtooth Slots
Parallel Slots
Contraction
Joint Galleries
ELEVATION 3650

Fig. 25.—Horizontal Sections of O'Shaughnessy Dam as Enlarged, Showing Arrangement of Slots Along Longitudinal Joint, and of Radial Contraction Joints, 150 Ft and 200 Ft Above Base of Addition

Concrete construction of the addition, to raise the crest 85.5 ft to its present elevation, was begun in 1936 and completed in 1937. Grouting of joints continued several months into 1938. Where the new concrete rests on the horizontal surfaces of the old base section and of the original crest, keying was provided by cutting notches in the old concrete, 16 in. wide, 8 in. deep, and 4 ft long, and by roughening the surface between notches, removing all of the original surface to an average depth of about $\frac{3}{4}$ in. The arrangement of notches was such that about 45% of the total area was notched and 55% merely roughened.

The treads and risers of the steps on the old downstream face were roughened, and in addition the risers, in the north one third and the south one third (approximately) of the face, were notched vertically. The surfaces of the siphon spillways were roughened.

The contraction joints of the old dam continue downstream into the new work, and a joint was provided in continuation of the aforementioned crack. Where the distance between the old joints exceeds 60 ft, the new section has intermediate joints, except in the conduit block. Some irregularities in spacing near the ends result from topographic considerations. Most of the blocks are 42 to 50 ft between joints, the minimum length of block is 30 ft, and the maximum 97 ft. The average of the 17 blocks along the 900-ft crest length is 53 ft.

The transverse joints of the original dam are keyed, with panels 15 ft wide, beveled at the edges, and continuous from near rock foundation to near the upper surface of concrete. This same arrangement was used in the new section, except that at each 50 ft of elevation the key is suppressed and a sheet steel horizontal grout stop spans the joint. Grout stops also were placed near the upstream and downstream limits of the joints.

In considering the joint along the downstream face of the original dam, the simple pouring of new concrete against the face, as a part of the pouring of each lift, was rejected at the start because of the obvious danger of developing undesirable shearing stress and opening cracks between the new concrete supported on the steps and the new concrete downstream which would shrink appreciably downward as cooling progressed in the lifts previously placed, and as compression increased in the lower concrete due to the increasing weight of the superposed concrete. The ideal would be to keep the new section out of contact with the old face until it was fully cooled, but this would be impossible because of the necessity of supporting the new concrete overhanging the old face. The design finally evolved provided a series of slots formed between the old and the new concrete, averaging about 5 ft in thickness normal to the face and about 20 ft in width measured along the face, extending continuously from the foundation of the addition to the old crest level. Adjacent slots were separated by walls or ribs 2 ft thick, poured integrally with the adjacent mass concrete. The downstream face of the slot was so formed that the concrete filling the slot would key the old and new sections to transmit stresses in radial planes. In horizontal section, the slots in the middle portion of the dam were rectangular; those toward the ends were tapered, with the broad ends toward the abutments; this latter arrangement, combined with the aforementioned notches in the risers of the steps of the old face, provided keying for transmission of horizontal stress.

The ribs between slots were intended to carry very high unit stresses during construction, and were expected to crack, as most of them did. Cracks, which in general were mere hairlines or little more, formed across the ribs, normal to the slope of the face, from the outer corners of most of the steps, but there was no indication that these cracks extended into the adjacent mass downstream from the ribs.

A further tie between the old and the new work was provided by placing 1¼-in. square deformed reinforcing steel bars across the slotted joint, normal to the general direction of the slope of the face. There is one horizontal row of bars for each 5-ft lift, and the average horizontal spacing is about 2.5 ft. The bars are grouted into holes drilled 5 ft deep in the old concrete, and extend through the slots and 5 ft into the new concrete downstream from the slots.

To drain the longitudinal joint, horizontal drains extend along the original face with a vertical spacing of 5 ft, connecting to gutters in inspection galleries.

All of the new concrete was cooled by circulating water through embedded pipes of 1-in. nominal size on 5.5-ft spacing, in the same manner as in Boulder Dam. The slots were filled with concrete after the adjacent mass concrete was cooled, and then the concrete in the slots was cooled.

Finally, all joints (except, of course, horizontal ones) were grouted. This operation included the radial joints in the original dam, as well as the slot and the radial joints in the new section. Most of the grouting was done after practically all concrete was completed.

Apparently the objects in view in the design have been achieved; data so far obtained indicate that the dam as enlarged is behaving as it would if it had originally been built to its present size.

Basically, in the case of O'Shaughnessy Dam, the design of the longitudinal joint against the original downstream face, with the slots, ribs, and tie bars, resembles that used in the first enlargement of Assuan Dam in Egypt. In that work steel tie rods 1¼ in. in diameter, spaced 1 m apart horizontally and vertically, were set about 4 ft deep in the old cemented rubble masonry to project an equal distance into the new, and a 6-in. slot was left along the original face. The overhanging new masonry was supported on the tie rods and on 6-in. ribs spaced 49 ft apart. The slot was filled by grouting through perforated pipes after allowing time for equalization of temperatures in old and new masonry. The ribs appear to have been intended primarily to control the flow of grout, their functioning as supports being incidental.

Barton M. Jones,[77] M. Am. Soc. C. E. (by letter).—The excellent paper by Messrs. Paul and Jacobs presents the important phases of foundation preparation in an interesting and enlightening manner, well covering the ground, and along general lines of preparation by repair, rather than by wholesale removal, of rock. Such removal becomes time consuming and very costly when the cost of the concrete to replace it is added to the cost of the rock excavation. The parts of the paper covering recent developments in equipment and methods are of special interest.

The paper calls attention to the necessity of controlling the pressure in grouting operations and cautions against pressures that are too high. However, pressure ranges of 50 to 200 lb per sq in. for low-pressure grouting and 250 to 600 for high-pressure grouting are mentioned. It is clearly stated that there is danger of rock dislodgment with these pressures and that the utmost caution must be exercised in their application. This danger should be fully comprehended by those engaged in grouting operations as otherwise serious damage may be done to the foundation. Of course, the direction and position of the seams, and character of the rock, dictate the safe allowable pressures. Little displacement or damage could be done to a foundation in which the seams are nearly vertical, whereas, on the other hand, horizontal seams must be grouted with great precaution to avoid lifting portions of the foundation, and thereby progressively extending the area of the seams, and possibly disturbing the adjacent grouting that has already been done in other seams. One of the principal objections to the occurrence of a slight uplift is that it may extend the area upon which the grouting pressure acts with a resulting accumulative uplifting effect.

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The writer desires principally to offer a general plea and some arguments in behalf of lower grouting pressures. It is safe to say that in many cases satisfactory grouting from the standpoints of bearing, uplift, and seepage can be accomplished at low and moderate pressures. The use of higher pressures may obscure the possibilities to be found in lower pressures. It is observable during grouting operations that most of the grout (possibly 99%) enters the grout holes and seams before the back pressure starts on its final upward jump. In other words, the high pressures and attendant hazards, whatever they may be, are related to a final spoonful of grout. The question may be asked, "Why use high and probably damaging pressures where low pressures can serve the purpose; and, once being convinced that the horizontal seams must, and can, be grouted with moderate pressures, just why should extremely high pressures be used on steeply inclined seams simply because the formation will permit?" There is reason for using high pressures in grouting deep and fairly tight seams in igneous rock formations where drilling becomes a larger part of the cost.

Uplift of horizontally stratified rock begins at surprisingly low pressures. For example, at Norris Dam it was the practice to grout the different seams in each hole separately, and the pressure in pounds per square inch was not allowed to exceed the depth in feet of rock over that seam. The advisability of this rule was verified by many observations of uplift. Grouting to a depth of 40 ft was done over the entire base of the dam and power house. Over this area of shallow grouting the pressures were kept under 35 lb per sq in., and the highest pressures used on the project, under the completed dam, were from 125 to 150 lb. The numerous seams in the foundation under the dam ranged from close contacts to open seams or pockets in which the rock was separated by a foot or more. The cost of drilling and grouting (approximately $759,000) indicates that the work was both extensive and difficult. During an experience record of three years it has been found that the treated foundation is particularly sound and tight against seepage under the 200 ft of head against the dam, and there has been no evidence to indicate that the relatively low grouting pressures used were in any way inadequate or that higher pressures would have been better.

Uplift generally exists to some degree during grouting, and the need is to keep it under observation. The frequent occurrence of uplift was brought to attention by a special type of uplift gage originated, perfected, and used extensively at Norris Dam. Owing to its simplicity and usefulness, a description of the gage might be of interest. In principle, it detects uplift at the surface with reference to the underlying undisturbed rock. In its elementary form the gage is constructed by drilling a hole within the area to be grouted and several feet deeper than the surrounding grout holes. A substantial steel rod is lowered into the hole and securely anchored into the underlying rock at its lower end with cement mortar, or otherwise. A substantial steel yoke, the under side of which just touches the top end of the rod, is securely anchored to the surface rock by cementing the ends of both legs into holes drilled about a foot deep. Any upward movement of the rock surface carries the yoke upward and increases the gap between it and the top end of the rod. Certain refinements and precautions become evident in its use. For instance, grout might enter the hole from the seams being grouted, and to keep the rod free from the side walls it should be surrounded with a soft wrapping or some kind of flexible tube. The equipment must be sturdy to resist the hazards of a construction area such as truck wheels. The yokes may be made of 2-in. steel bar, and smooth brazed points will furnish rustless gaps between the yoke and top end of rod that can be measured with feeler gages to within 0.001 in., thereby providing accuracy with the necessary ruggedness and permanency while avoiding the many disadvantages of delicate indicator or other gages. Temperature changes in the rod are of no consequence in grouting operations, but for some other uses of the gage, such as measuring settlement of turbine foundations or foundation deflections under a dam, an allowance may be necessary to correct for the effect of the seasonal temperature variation on length of the rod.

The gages form a most useful aid in eliminating conjecture in controlling grouting operations. In using them, several should be located in the area to be grouted, and it is an easy matter to check and record the uplift each hour or oftener by simply measuring the gaps between the ends of the rods and the yokes.

The knowledge obtained from uplift gages cannot be put to full use unless the grouting is controlled and the pressures are carefully limited to the values found safe for each area or group of holes. It is not safe to depend upon hand control of the pressure when reciprocating pumps are used for grouting. With this very satisfactory type of equipment it becomes necessary to have a dependable adjustable pressure regulator on the power end of the pump, and an adjustable pressure-relief valve on the grouting pipe. Sometimes it is also necessary to guard against excessive pressures when washing out seams preparatory to grouting.

In conclusion, it may be stated that on extensive or important grouting operations there is justification for continually checking the pressures that may be used with safety, and then to keep within those pressures by means of automatically regulated equipment. Rather than to exceed the safe grouting pressures in an attempt to force in more grout, a better procedure would be to use more holes. The ultimate cost may be less as much damage and further grouting may be avoided. It is believed that this discussion is a clarification and amplification of principles already set forth on grouting in the most excellent paper by Messrs. Paul and Jacobs.

P. E. GISIGER,[78] M. AM. SOC. C. E. (by letter).—During the past twenty years greater progress has been made in the art of designing and building large dams than during centuries before. The paper on the fundamentals of dam design by Messrs. Houk and Keener, who belong to a group of engineers responsible for a very large part of this progress, is, therefore, unusually authoritative. It is particularly valuable for those engineers who are not intimately and actively connected with the building of high dams because it permits them to visualize what has been accomplished and to what extent it is now possible to substitute exact reasoning in problems, which, a relatively short time ago, were left to guesswork or to the care of the so-called safety factors.

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In 1935 an article was published in French[79] by Henri Juillard, which affords an interesting comparison of the most advanced thought on the subject in Europe and in the United States. It is believed that this article is valuable enough for a short digest of it to be presented as part of the discussion of this paper as well as of the Symposium in general.

Mr. Juillard confines himself to dams of the straight gravity and curved gravity type. He first questions the idea still held in some quarters that the massive straight gravity dam is the safest type of structure. The two points sometimes argued in favor of the straight gravity dams—(*a*) the possibility of a simple analysis of all forces and stresses, and (*b*) the absolute stability on account of the ever-present mass—are shown to be mistaken. The first contention is demonstrated to be faulty mainly on account of the twist action resulting from sloping abutments, but also because it could be rigidly correct only for the conditions of homogeneous material, uniform temperature, and foundations with elastic properties equal to those of the dam, all of which are never fully realized. The second contention is accepted for dams of moderate height, where stresses are not a factor of much importance. A high dam, however, might fail from shear or compression at its toe even if its weight were ample for external stability. In the final analysis, therefore, the safety of a high dam does not depend upon its mass, but upon the stresses that are developed, and can be resisted by the material of the dam itself and the underlying rock. From this it follows that no type of dam can be declared the "best" or "safest," but that for a given location one type may be better adapted, and therefore safer, than another type.

Mr. Juillard then states that originally a straight gravity dam with a width-height ratio of 85% was proposed for a high dam in Switzerland (the Spitallamm dam of the Grimsel development) but that finally a curved gravity or massive arch dam with a width-height ratio of 60% was adopted. It is emphasized that the main reason for this change was not the saving of about 130,000 cu yd of concrete, but the conviction that stress conditions would be more favorable for the curved dam.

Discussing concrete behavior during the setting and cooling period, Mr. Juillard emphasizes the importance of plastic flow of fresh concrete because without this property the various superimposed causes of volume change, such as heating, cooling, drying, and shrinkage, would result in an excessive number of cracks. Cracks are not quite unavoidable, but by proper subdivision into blocks and sequence of placing, their number can be substantially reduced, which, of course, is now recognized everywhere.

With regard to concrete mixes the importance of the proper proportion of fines in the aggregate is stressed. It is claimed that either a deficiency or an excess of fines will produce porous concrete which does not resist freezing, and that with the proper quantity of fines the least quantity of water will be required to produce workable concrete.

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[79] "Progress in the Construction of Large Masonry Dams," by Henri Juillard, *Le Génie Civil*, August 24, 1935, p. 178.

Mr. Juillard declares himself in favor of especially rich facings for the upstream as well as the downstream face of a dam. This is apparently not in agreement with most recent American practice and may be open to question with regard to the importance of having homogeneous material throughout the dam, apart from the economical side of this point.

The most impressive part of Mr. Juillard's paper is the final third wherein the need for continuous observation and inspection of dams after completion is discussed. It would seem natural that the investment represented by a large dam should call for close and constant check of its condition and behavior. It is rather astonishing, therefore, how long it has taken before the need for such supervision was recognized, and before the attitude was overcome that a masonry dam, after the last batch of concrete has set, is an absolutely stable, immovable, and noncorrodible object which does not require any further care.

The feature to which attention was given first, and for a long time almost exclusively, was leakage, especially if it amounted to more than had been expected. In the more recent dams, which have inspection galleries, intercepting tunnels, riser pipes, and other devices, it is easier to get an idea of the probable total leakage than in older dams where such devices are usually absent and the total loss of water must be judged from what comes to daylight at the downstream side.

In some dams devices have been installed which record the quantity of leakage water passing a given point. Mr. Juillard mentions apparatus which, in addition, records the percentage of lime carried in solution by this water. The latter point is emphasized to illustrate the fact that concrete is not an inert material, since a considerable tonnage of lime can be removed annually from a large dam.

Apart from cases of extreme leakage or very poor concrete, however, judgment on the continued structural soundness of a high dam must be based mainly on observation of its deformation under load and the influence of temperature. This may have been recognized quite early, but the practical difficulties of measuring deflections on a dam are much greater than, for instance, measuring deflections of a bridge. Therefore, such measurements are a fairly recent development.

The methods of measuring deformations used in Europe were described by Mr. Juillard in a paper[80] read before the World Power Conference in Stockholm, Sweden, in 1933. In the paper cited herein[79] he stresses the necessity of continuous, regular observation for the purpose of gaining complete knowledge of the factors affecting a dam and the availability of apparatus permitting the making of such observations at relatively little expense. He cites as an example the previously mentioned Spitallamm dam of the Grimsel development where the horizontal movement of a number of points is observed twice weekly by means of pendulum indicators, permanently built in, which give an accuracy of $\frac{1}{20}$ mm.

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[80] "Report No. 12" by Swiss National Committee, 1st International Cong. on Large Dams, Stockholm, 1933.

It is also stressed that these measurements, as well as others on recent dams, indicate a complete elastic behavior under load, contradicting some earlier opinions based on incomplete observation to the effect that dams undergo a permanent deflection. It is important, of course, to separate the effects of temperature from those of load. Temperature effects predominate near the crest of a dam and are subject to daily change. In 1939, Douglas McHenry and Roy W. Carlson, Assoc. M. Am. Soc. C. E., presented[81] results of observations on the Norris Dam which illustrate this point and indicate the fact that Mr. Juillard's ideas are in agreement with recent practice in the United States.

Mr. Juillard goes on to state that, although the measurements on the dam proper are those of most importance, the rock under and next to the dam must also be considered. A large number of complex questions remain to be solved in this respect. It is not known, for instance, what part of the deformations caused by the weight of the dam as well as the stored water is permanent or elastic, how this may change with different kinds of rock, how changes in surface temperature due to the covering with water affect the rock, how deep these influences go, and how long it takes to establish a new equilibrium.

Every one of these uncertainties is an additional reason for continuous, detailed observation of an important dam after its completion. The proper safeguarding of investment should be reason enough for the relatively small outlay necessary. A much weightier reason is the safeguarding of life and property. In this respect, it may be reasonably claimed that the few disasters which have occurred could have been greatly mitigated, if not altogether avoided, if regular, frequent observations of structural movement on those dams had been taken.

Finally, Mr. Juillard stresses that only continuous observation on the completed structures will indicate whether the design assumptions and methods were correct and will stand the test of time and that only by comparison of actual behavior with design theory can further improvements be expected in the efficiency of design and perfection of material.

The foregoing tends to show that ideas on the design and construction of large masonry dams have, in recent years, moved toward the same conclusions in Europe as well as the United States. Twenty-five or thirty years ago masonry dams of European design showed a tendency to conform to a set type, without much regard to site conditions. They were usually dimensioned to satisfy rigid safety requirements (such as the "Lévy condition") based on strictly two-dimensional stress analysis and contained often elaborate expansion joint and drainage provisions. In America, on the other hand, development went forward without much benefit or hindrance by precedent. Uplift assumptions, which largely govern the mass of a gravity structure, and provisions for drainage and treatment of joints were generally left to the judgment of the designing engineer and showed great variation.

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[81] "Measuring Dam Behavior," by Douglas McHenry and Roy W. Carlson, *Engineering News-Record*, March 30, 1939, p. 58.

With the increase in number and size of dams that were designed and built since 1920, research into the design problems was greatly stimulated and brought about a vastly increased knowledge about the way in which the external forces acting on a dam affect its internal stresses and deformations. This general clarification of ideas produced more uniformity of design. Mr. Juillard's paper, summarizing the same period of development in Europe, shows that at the present time the basic considerations as well as methods of design on both sides of the Atlantic are practically the same. The importance of twist action caused by sloping abutments, the combination of cantilever and arch action in curved gravity or massive arch dams, and the influence of elastic deformation of the underlying rock are given the same relative degree of emphasis by Mr. Juillard as by the most authoritative American thought which is embodied in the papers presented for this symposium. Beyond that the writer believes that Mr. Juillard's insistence on periodical and systematical check of the behavior and performance of a dam after its completion is an important and valuable contribution to the subject matter presented on this occasion.

JOSEPH A. KITTS,[82] ESQ. (by letter).—As an index of the present state of progress in concrete manufacture and control as applied to dams, the paper by Mr. Tyler comprehends only a very recent school of thought on the subject, as indicated by its appended list of references. Although it admits that there are some scientific questions yet to be solved by this school, it says (see "Synopsis") that "knowledge concerning properties and behavior of concrete * * * has been enormously increased during the last few years," being in complete accord, on this last point, with each succeeding report of the kind in the past.

On the other hand, the concrete manual mentioned in the Appendix (2) shows that this school of thought has very little appreciation and knowledge of the fundamental physics and basic principles of concrete mixtures brought forth in twenty-two centuries of laborious and costly research since about 250 B.C., when Archimedes discovered how to determine the absolute volume (the basic measure) of an irregular solid by liquid displacement. Instead, this school has adopted a false physics using weight as the basic measure. Its false physics has led to erroneous test results. Its erroneous test results indicate a new and simplifying principle of mixtures, although definitely and widely shown to the contrary 40 years before.[83] It adopts this pseudo-principle as its "fundamental rule." Its false physics and false principles are written into its specifications and naturally affect the practical features of control. The result has been an enormous waste of cement and aggregate, and the quoted "enormous increase of knowledge concerning the properties and behavior of concrete" is based on, and colored by, this false physics. Mr. Tyler does not offer anything to correct the situation indicated in the foregoing, nor does he appear to appreciate it.

In a series of seventeen articles,[84] the writer has coordinated all the known and authoritative fundamentals of concrete mixtures in complete detail of mathematical analysis and expression, giving sources, authorities, and practical application. The absolute volume measure of Archimedes was used as the only logical basis on which to coordinate: (*a*) The matrix-to-voids principle of François Coignet;[85] (*b*) the absolute volume composition law of René Feret;[83] (*c*) the grading principle of Sanford E. Thompson and the late William B. Fuller,[86] Members, Am. Soc. C. E.; (*d*) the water-cement strength law and fineness modulus principle of Duff Abrams,[87] M. Am. Soc. C. E.; (*e*) the cement-content strength law and fineness modulus law of the writer;[84] and (*f*) various miscellaneous principles by others. These are the complete fundamentals of concrete technology; there are no successful or basic alternatives. With this history of concrete research and the coordination of basic principles of concrete mixtures before them, the sponsors of Mr. Tyler's school of concrete control strangely went out of the way to adopt false physics and the disproven theory that water content and slump remain constant as the cement content is increased.[83]

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[82] Cons. Engr. (Joseph A. Kitts Co.), San Francisco, Calif.

[83] *Bulletin de la Société d'Encouragement pour l'Industrie Nationale*, by René Feret, Vol. II, 1897, p. 1593.

[84] "Coordination of Basic Principles of Concrete Mixtures," by Joseph A. Kitts, *Concrete*, November and December, 1931; April, 1932, to April, 1933; and February to April, 1934.

Absolute Volume the Basic Measure.—"The absolute volume is the basic measure of concrete ingredient proportions and characteristics." That is the fundamental law of concrete physics and technology.[84] Cement, aggregate, and concrete tests and concrete research, disregarding this law, have been almost worthless, and this disregard accounts for the fact that man's 10,000 years of concrete making had not evolved a complete concrete technology until the discovery of the cement-content strength laws, in 1929. Until this fundamental law of the physics of concrete materials and mixtures is used, there can be no concrete technology worthy of the name; and, until then, research of the properties and behavior of concretes will continue to be largely barren waste and the source of costly errors.

Weight is the most accurate and flexible means of measuring concrete ingredients, and should be required for the measurement of the solid ingredients of concrete for major structures, and for structural concrete in general. It is an amazing fact of concrete history, however, that, where the volumetric batcher, basket, barrow, or bin is used as the means of measure, the volume of the measure (not the absolute volume of material in the measure) has been the basis of proportions, and is so today, on 75% of all concrete specified. As in the case of volume as the means of measure, when weight has been adopted as the means of measure, it has also been adopted as the basis of measure.

Confusion of Measures.—Mr. Tyler treats almost all of the particulars of concrete control except the fundamental physics of materials and mixtures. The writer contends that there can be no sound understanding of any feature of concrete control until the fundamental physics of materials and mixtures is understood; and the fundamental physics cannot be understood until the basic measure is recognized. Accordingly, the confusion of measures appearing in sources cited in the Symposium should be considered. It should also be noted that centuries of civil engineering thought, or lack of it, are responsible for this confusion.

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[85] "Coignet Béton," by the late Q. A. Gillmore, M. Am. Soc. C. E., Washington, D. C., Government Printing Office, 1871.

[86] *Transactions*, Am. Soc. C. E., Vol. LIX (1907), p. 67.

[87] "Design of Concrete Mixtures," by Duff A. Abrams, *Bulletin No. 1*, Structural Materials Research Laboratory, Lewis Inst., Chicago, Ill., 1919.

In this school of thought the water-cement ratio is expressed by weight instead of volume of water to absolute volume of cement. Thus, a water-cement ratio of 1.0 by weight means a water-cement ratio of 2.8 and 3.2 by volume, for cements having specific gravities of 2.8 and 3.2, respectively. Obviously, the weight basis is indefinite. On the other hand, volume of water to absolute volume of cement is a definite ratio and conception, regardless of the specific gravity of the cement, and engineers know that 1 cu ft of water added to 1 cu ft absolute volume of cement, for example, contributes the sum, or 2 cu ft, to the concrete mix.

Fineness of cement is expressed as specific surface, or square centimeters, per gram of cement, instead of square centimeters per cubic centimeter absolute volume of cement.

Fineness modulus of aggregate is determined from the proportionate weights retained on the sieves, disregarding natural differences in the specific gravities of the fine and coarse parts of the aggregate.

Unit weight of aggregate is determined as the weight of dry-and-rodded material filling the standard container of unit volume. The volume of the container is the basis of measure, not the absolute volume of material contained.

Absorption and moisture contents of aggregate are determined as weight of water to weight of aggregate (0.02 absorbed water by weight, in the case of Haydite aggregate having an apparent specific gravity of 1.0, is 0.02 cu cm of water per cubic centimeter absolute volume of Haydite, and is 0.053 cu cm of water per cubic centimeter absolute volume of rock having a specific gravity of 2.65).

Specific gravity of solids is determined as weight of solid to weight of water displaced by the solid, or, weight of solid in grams to absolute volume of solid in cubic centimeters. The basic measure of the material is the absolute volume. This is the simple physics developed by Archimedes some twenty-two centuries ago.

One sees, then, as general and standard practice, the indiscriminate use of loose-moist, loose-dry, and dry-rodded bulk volume, and the volume of the container, weight, and the absolute volume as entangled bases of proportions and characteristics. This is not physics, science, engineering, or logic.

Using weight, the gravitational characteristic, as the basic measure of proportions to produce an absolute volume of concrete, when it is known that the absolute volume of concrete is the sum of the absolute volumes of the ingredients, seems too obviously unscientific for an engineer to accept. On the other hand, since absolute volume has not been accepted as the basic measure of proportions in twenty-two centuries, perhaps it is too abstruse an idea for the layman in concrete physics and technology to grasp. Another point of view is that concrete manufacture is a simple and practical art, and not a complex science. The writer observes, however, that the science works —practically, efficiently, and economically; the simple and practical art does not. Mr. Tyler's point of view, as stated in his "Conclusion," is that "The art of building concrete dams is becoming a science." Lack of appreciation of the absolute volume as the basic measure of concrete proportions is still holding it in leash as it has concrete technology for twenty-two centuries.

False Theories Due to False Physics.—Under the heading, "Concrete Control: Proportioning," Mr. Tyler states that "water content of concrete at fixed workability changes little with cement content." In other words, with a given mix, as aggregate is replaced with an equal absolute volume of cement, the slump, consistency, or workability of the mix remains the same, or changes little, according to Mr. Tyler.

In 1897 the noted French engineer and dean of concrete physicists, René Feret, published the results of an elaborate series of tests, made by him, of fine, medium, and coarse sand-cement mortars of normal consistency, in which he varied the absolute-volume cement content in ten stages from about 3% to about 53% by volume of the resulting mortar.[83] A table of the results can be found in a publication by Frederick W. Taylor and Sanford E. Thompson.[88] The water per unit volume of concrete for Feret's fine sand "*D*" follows the semilogarithmic equation

$$\log W = 0.33\,C - 0.58 \text{ (approximately)} \ldots\ldots\ldots\ldots (7a)$$

in which W is the volume of water per unit volume of concrete, and C is the absolute volume of cement per unit volume of concrete. His coarse sand "*G*" follows the straight-line equation

$$W = 0.58\,C + 0.1 \text{ (approximately)} \ldots\ldots\ldots\ldots (7b)$$

Feret's medium sand lies between the two foregoing, and their three graphs approach neat cement mortar composed of: 0.534 cement, 0.414 water, and 0.052 air. Thus, at 5% cement, the water content ranges from 13% for coarse sand to 27% for fine sand; and, at 53% cement, or neat mortar, the water is 41%; that is, the water content for neat cement mortar of a given consistency is shown as 52% to 215% greater than that for a sand-cement mortar containing 5% absolute volume of cement (2.75 sacks per cubic yard).

The writer has made concrete and neat cement mortar tests of Bonneville type cement which show the relations of cement and water contents of mixtures of a given slump, as indicated in Table 14.

In changing from 1 bbl of cement per cubic yard of concrete to neat cement mortar, it is seen by Table 14 that the water content for a given slump is increased 257% for the 1-in. slump and 131% for the 8-in. slump. Any other calcareous cement will show values of a similar order.

Table 14, for 1½-in. maximum size of aggregate, confirms Feret's tests of mortars made 40 years earlier. Mr. Tyler's qualification that the water changes little with cement content is quite true if the cement content is changed only a little; concrete manufacture, however, comprehends cement contents from 2% to 55% of the concrete volume. The false theory is due to test errors characteristic of the false physics of using weight as the basic measure of ingredient proportions and characteristics.

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[88] "Concrete, Plain and Reinforced," by Frederick W. Taylor and Sanford E. Thompson, 2d Ed. (1909), pp. 136–137.

Laws of Proportioning Are Needed for Any Maximum Size and Grading of Aggregate, Strength, and Slump of Concretes.—In 1931, the writer analyzed the water and cement content relations shown by data of the Portland Cement Association.[89]

TABLE 14.—WATER CONTENTS FOR NEAT CEMENT MORTARS COMPARED WITH THOSE FOR LEAN CONCRETES

Standard slump, in inches	CONCRETE, 1½-IN. MAXIMUM		NEAT CEMENT MORTAR	
	Cement	Water	Water	Cement
	Proportionate Absolute Volume			
1	0.073	0.118	0.422	0.549
2	0.073	0.124	0.432	0.542
3	0.073	0.132	0.442	0.536
4	0.073	0.140	0.452	0.529
5	0.073	0.151	0.462	0.522
6	0.073	0.163	0.472	0.515
7	0.073	0.181	0.484	0.507
8	0.073	0.216	0.499	0.496

For any given maximum size of aggregate and slump of concrete, the change of water content with change of cement content was discovered to follow the semilogarithmic equation,

$$\log W = m\,C + k \qquad (8)$$

(in which m and k are constants depending on the maximum size and fineness modulus of aggregate and the slump of concrete), provided the fineness modulus of grading follows the equation:

$$f = a \log C + b \log D + c \qquad (9)$$

in which D is the maximum size of aggregate and a, b, and c are constants.

Although there are occasions in which Eq. 8 is convenient to use, the constants m and k for a given maximum size or slump do not vary in accordance with any simple mathematical expression as the maximum size or slump is varied.

In 1929, the writer discovered that the law of the cement content for any strength, for a given maximum size and slump, is expressed by the equation

$$\log C = d\,S + e \qquad (10)$$

in which S is the compressive strength at a given age and d and e are constants depending on the maximum size and slump. It was further found that e, in Eq. 10, varied as the slump, or

$$e = g\,\text{slump} + h \qquad (11a)$$

and as the log of the maximum size, or

$$e = n \log D + j \qquad (11b)$$

in which g, h, n, and j are constants.

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[89] "Design and Control of Concrete Mixtures," Portland Cement Association, 2d Ed., January, 1927.

This family of cement-content laws is supplementary and complementary to the Abrams water-cement ratio law; it is closely accurate for strengths greater than 1,000 lb per sq in. at 28 days and indicates excess cement at desired strengths less than 1,000 lb per sq in. It is too well founded and too adequate to have been abandoned for a new theory which the competent tests of the dean of concrete physicists had shown to the contrary, 40 years before, and which the comprehensive data herein cited[89] had shown to the contrary more than 10 years before.

Theoretical Grading and Proportioning.—Mr. Tyler states that "Too close adherence to theoretical gradings usually leads to undersanded and harsh concrete mixtures," and "It is seldom if ever possible to calculate an aggregate grading and proportion a concrete mix without experimental data or actual trial batches of concrete as a guide." There are great differences in the shape of particles of crushed rock from various sources, of course. Angular, elongated, and lens-shaped particles demand a lower fineness modulus of grading (and more cement) than do rounded gravels. However, a grading suitable for natural gravel from one source will almost invariably be found suitable for gravel from any other source, provided the grading is on the basis of absolute volume measure. If theoretical gradings produce undersanded, oversanded, and harsh mixtures, and mixtures which segregate badly, the criteria, theory, or technique of grading is obviously wrong.

There are considerable data, of normal standard portland cement and crushed rock and gravel concretes, which supply good criteria of the principles of grading and proportioning as a basis of experimental work on any new project. Knowing the physical characteristics of the materials and the principles involved, gradings and mixtures can be calculated on the basis of the criteria with a high degree of certainty as to resulting workability, slump, yield, and strength. Not knowing the physical characteristics and the principles involved, the only recourse is trial mixtures and experiment, which, in turn, depend upon principles and technique. Of course, the physical characteristics of the materials are not fully established until they are tested in the concrete mixtures, and no dam concrete should be produced without preliminary tests and subsequent laboratory control on the job. Laboratory control means continuous testing of material characteristics, and occasional and sometimes frequent changes of the measured proportions, to maintain the basic structure of the concrete composition in absolute volumes of cement, water, and stone particles of various diameters, as physical characteristics of the materials vary. Sometimes, also, changed conditions require modifications of the basic structure of the concrete composition; but to say that "It is seldom if ever possible to calculate an aggregate grading and proportion a concrete mix without * * * trial batches * * * as a guide" points to one or a combination of the following causes of failure: (1) Wrong criteria, (2) false physics, (3) false theories, (4) wrong characteristics, (5) graphical approximations, (6) erroneous equations, (7) erroneous technique, and (8) the major source of failure in grading, proportioning, and control is lack of appreciation of absolute volume as the basic measure of ingredient proportions and physical characteristics.

Many Types of Grading Are Practicable.—The average pit run, although generally somewhat oversanded, may be the most practicable and best grading from the view of engineering economy. If merely 5% of the material is finer than the No. 100 sieve and the remainder is sound, there is generally little reason for wasting any of it. If there is 55% finer than the No. 100 sieve, there would then be justification in wasting 50% of the pit run, since such excess of fine sand is merely an adulterant to the water-cement mortar, reduces the strength and density of the concrete, and generally costs less to waste than to use. There have been many engineering crimes committed in the name of grading, at added expense and sometimes with loss of quality of the concrete. "Soaking" the contractor to produce 5,000-lb concrete by wasting good aggregate, when the specification merely calls for 2,500-lb strength, is a practice which automatically increases the contract price with little benefit to the owner. From the point of view of national economy, it would appear to be better engineering economy to allow the contractor to have an extra million dollars of profit in his pocket rather than have it poured out and forever lost in an enormous and unwarranted waste pile.

The ideal grading of the coarse aggregate from $\frac{D}{10}$ to D is no doubt a straight line as proposed by Messrs. Fuller and Thompson, in 1907. An ellipse from the No. 100 sieve, or 0.0059 in. to $\frac{D}{10}$ and tangent at $\frac{D}{10}$ to the straight line from $\frac{D}{10}$ to D, is perhaps the ideal sand to go with the ideal coarse aggregate. The equation of the ellipse is

$$y^2 = \left(\frac{b^2}{a^2}\right)\left[a^2 - (a - x - 0.0059)^2\right] \ldots\ldots\ldots\ldots (12)$$

in which

$$a = \frac{y'\,(0.1\,D - 0.0059) - m\,(0.1\,D - 0.0059)^2}{y' - 2\,m\,(0.1\,D - 0.0059)} \ldots\ldots\ldots (13a)$$

$$\frac{b^2}{a^2} = \frac{(y')^2}{\left[2\,a\,(0.1\,D - 0.0059) - (0.1\,D - 0.0059)^2\right]} \ldots\ldots\ldots (13b)$$

and x is the sieve aperture, y is the percentage passing the given sieve, y' is the percentage passing at $x = \frac{D}{10}$, m is the slope of the straight line expressed as a percentage, D is the maximum sieve size of the aggregate, and a and b are constants for the given maximum size and point of tangency. This is very simple to use when one becomes acquainted with it, and gradings of any given fineness modulus can be determined without much trouble.

The Bolomey Curve[90] is simply a modified parabola of the proportion passing and can be determined for any fineness modulus by varying the value of N in the equation for the proportion retained

$$r = N\left(1 - \sqrt{\frac{d}{D}}\right) \ldots\ldots\ldots\ldots\ldots\ldots (14)$$

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[90] "Contrôle des Qualités des Ciments," by J. Bolomey, *Travaux*, Vol. 21, No. 54, June, 1937, pp. 256–265.

in which r is the proportion retained on the given sieve of d opening, and D is the maximum sieve size of the aggregate.

The Talbot Curve described by Mr. Tyler (Eq. 4) should be written in the form:

$$r = 1 - \left(\frac{d}{D}\right)^n \qquad (15)$$

in which n is an exponent, and not a "factor" as stated.

The Fineness Modulus Is the Practical Tool of Grading.—The graphical method of determining grading proportions is obsolete, slow, tedious, and inaccurate. By use of the fineness modulus and tabular values of the particular grading curve used, the proportions of 3 and 6 aggregate parts can be calculated in 3 and 6 min, respectively. Slaves to inflexible graphs are obviously rule-of-thumb workers in concrete technology. The graph as an illustration of the equation is useful, of course, but its use as the only means of determining values gives it a fictitious importance which generally overlooks the principles of variation involved. Eqs. 4, 12, 13, 14, and 15 each must comprehend a large number of curves whose fineness moduli need to correspond to Eq. 9, depending upon shape and maximum size of aggregate, and upon the cement content. For rounded gravel aggregate, the fineness modulus of the grading curve (see Eq. 9) should follow the equation

$$f = 1.1 \log C + 2.75 \log D + 2.31 \qquad (16)$$

in which C is pounds of portland cement per cubic yard and D is the maximum square-hole size of the aggregate. If crushed stone or slag is used as coarse aggregate, the value of f should be reduced 0.25, and 0.40 if the crushed material consists of unusually flat and elongated particles. If stone screenings are used as fine aggregate, the value of f should be reduced 0.25. Other conditions may require a reduction of 0.50 and, again, may permit an increase of 0.25 or more.

The fineness modulus of aggregate, as a means of grading and proportioning, was presented by Professor Abrams[87] in 1918, and offered a more accurate and more expedient method than the tedious graphical one. Its sponsors in 1918, however, rejected it in 1928, instead of perfecting the technique of its use. As a consequence, the value and utility of this long-needed and essential physical characteristic of an aggregate has not been generally appreciated. Too little attention has been paid to the fact that the fineness moduli of the parts of an aggregate between the standard sieves in Col. 1 following are as shown in Col. 2:

Column 1	Column 2	Column 1	Column 2
No. 100–No. 50	1	$\frac{3}{16}$ in.– $\frac{3}{8}$ in.	6
No. 50–No. 30	2	$\frac{3}{8}$ in.– $\frac{3}{4}$ in.	7
No. 30–No. 16	3	$\frac{3}{4}$ in.– $1\frac{1}{2}$ in.	8
No. 16–No. 8	4	$1\frac{1}{2}$ in.– 3 in.	9
No. 8–No. 4	5	3 in.– 6 in.	10
		6 in.–12 in.	11

Accordingly, the fineness moduli of the parts of a theoretical or actual grading curve from 0–$\frac{3}{16}$, 0–$\frac{3}{4}$, 0–$1\frac{1}{2}$, or $\frac{3}{16}$–$1\frac{1}{2}$, etc., can be calculated readily.

If it is desired, for example, to combine normal ¾-in.–1½-in. and 1½-in.–3-in. job aggregates for a straight-line grading, it is evident that, theoretically, the ¾ in.–1½ in. should be 33%, and the 1½ in.–3 in., 67%; and the theoretical fineness modulus of the ¾ in.–3 in. is, therefore, (0.33 × 8) + (0.67 × 9), or 8.67. If the actual fineness moduli of the job aggregates are 7.95 and 9.07, the job proportions by absolute volume are determined by simple proportion (as shown by Professor Abrams[87]): $\frac{K - T}{K - F} = \frac{9.07 - 8.67}{9.07 - 7.95} = 0.357$ of ¾-in.–1½-in. aggregate, and $1.0 - 0.357 = 0.643$ of 1½-in.–3-in. aggregate.

With this key, its elaboration should be obvious to any average mathematician familiar with the most simple operations of algebra. It can be used with the grading of the average pit run or with the most elaborate grading equation. The fineness modulus is a characteristic of the whole and the parts of any grading equation and is complementary to it. It is a physical characteristic quite as important in its utility as the specific gravity. Concrete technology would not be complete without it. Concrete control demands its constant use for efficient control.

Ample Slump Is Important.—Mixtures can be too dry for the conditions of use. Dry mixtures of 3-in. slump, and less, trap air and require an excessive amount of puddling work, and the volume of honeycomb increases with the dryness. Dry mixtures are the most permeable, medium mixtures the least permeable, and wet mixtures are less permeable than dry mixtures. The tendency since 1918, on dams and elsewhere, has been to make concrete too dry. Less than 3-in. slump is not warranted by the theoretical but questionable improvement of quality, nor by engineering economy. Making concrete so dry that it will not flow in chutes on 40% grade is abandoning the practical and economical for a questionable theory which is more costly. Experimentation too often looks at the quality obtained under laboratory conditions without regard to the quality per unit of cost on the job. Laboratory results too often disregard the labor and time element. During all the history of concrete there have been three schools of consistency: The Drys, the Wets, and the Moderates. The "old army specification" was "the consistency of fresh cow dung," as recommended[91] by the late Gen. Q. A. Gillmore, M. Am. Soc. C. E., about 1863. The terrible examples of honeycombed and patched concrete, since the inception of the Abrams water-cement ratio-strength law, suggest that practical lessons from a cow should supplement a theoretical knowledge of the Abrams law.

Since 1924 the writer has made many observations of the slumps and other conditions of mixture which produced satisfactory workability. The "optimum slump" for hand or manual puddling, tamping, and hammering (and flow in metal chutes on 40% grade) was found to be as given in Table 15. Mechanical vibration will permit 1 in. less slump in the case of narrow and difficult sections in structural concrete, 3 in. less in massive concrete, and 4 in. less in pavements; but a reduction to less than 3-in. slump is a source of increased and unavoidable defects and increased cost. In general, concrete should be not only plastic but fluid, for the optimum results. It should flow in a smooth, metal, parabola-shaped trough on a 40% slope (1 on 2½).

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[91] "On Limes, Hydraulic Cements and Mortars," by Q. A. Gillmore, *Professional Paper No. 9*, Corps of Engineers, U. S. Army.

Conclusion.—Much more should be said regarding the practical considerations covered by Mr. Tyler's paper, as well as the scientific and technical.

TABLE 15.—OPTIMUM SLUMPS

Maximum size (inch) (square aperture)......	3/16	3/8	3/4	1½	3	4½	6
Optimum slump (inch)......................	8.6	8.4	8.0	7.3	6.0	5.0	4.1

Mr. Tyler's school of concrete control is producing most excellent concrete, without doubt; but, also without doubt, that school of thought is using false physics, false theories, and practical ideas colored by them, at great expense and waste. No one person is responsible: The profession, the civil engineering colleges, the Society, every individual engineer—each is equally responsible for the chaos which has existed and still prevails in the accepted knowledge of concrete physics and technology. Until the fundamental physics of material characteristics and the laws of concrete mixture composition and quality are understood, there can be no prideful progress in concrete control on dams or elsewhere.

S. O. HARPER,[92] M. AM. SOC. C. E. (by letter).—The summarization, by Mr. Steele, of the practice in the art of building concrete dams, is a valuable contribution to the literature on dams. The painstaking care evident in the assembly of Table 11 and the supporting comments in the Appendix make of that section a most valuable reference. Comments made herein are listed under the same headings as appear in the paper.

Joint Spacing.—The spacing of vertical contraction joints so that joints occur at sudden changes in the foundation profile in order to minimize the formation of cracks, as discussed by Mr. Steele, is theoretically sound but usually impractical for various reasons. In the first place, the foundation conditions cannot be predicted with sufficient accuracy, prior to actual uncovering, for predetermination of the spacing of joints such that they will occur at the foundation breaks. Secondly, the usual irregularities encountered are seldom in directions or on straight lines that lend themselves to the location of contraction joints at the points of irregularity. Furthermore, the additional cost of variable size in forms as compared with uniform dimensions grows to a sizable value when account is taken of the construction delays and extra operations imposed by nonuniformity in procedure. In many cases material improvement can be effected in irregular foundations by a little expenditure for additional excavation. The undesirable effects of uneven foundation conditions in contributing toward cracking can be greatly minimized by suitably placing concrete in the depressions to build up the foundation to a general level and by adequate temperature control of concrete so placed in advance of general construction.

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[92] Chf. Engr., U. S. Bureau of Reclamation, Denver, Colo.

Slots Versus Grouted Joints Versus Open Joints.—The conclusion of Mr. Steele that the slot method of joint construction might be more satisfactory than the grout-film method is further substantiated when considering the normal variation in temperature of the concrete in thin dams constructed in extremely cold climates.

Very meager data are available of actual measurements of such temperature variations. Eight embedded resistance thermometers located across two transverse sections of Seminoe Dam in Wyoming, 21 ft from the top of the dam where the thickness is 21 ft, varied from an average of 24° in midwinter to 68° in midsummer. The mean annual air temperature at this location is 41°.

The contraction joints of Seminoe Dam were grouted in the early spring, as soon as the measured temperatures in the interior of the dam were more than 32°. Although every effort was made to grout the joints at the most favorable time, the average temperature of the eight thermometers at the top of the dam was 45° by the time this region was grouted. A reasonable estimate of the minimum closure temperature where contraction joints are grouted is about 40°.

Estimates of the range in temperature of a thin arch dam being considered for construction in a region in Colorado where the mean annual air temperature is 32° indicate that where the thickness of the dam is 20 ft the average concrete temperatures will vary from 5° to 53°. Severe stresses would be imposed upon this arch subjected to maximum water load during periods of minimum temperature if closure were made at 40°. It is estimated that closure could readily be made with slots with mean concrete temperatures as low as 25°, which, in this instance, would be a considerably more favorable closure temperature and would obviously be difficult to accomplish with grouted joints.

Bonding Joint Surfaces.—Mr. Steele's statement that "The adequate bonding of horizontal and inclined joint surfaces has been one of the most difficult problems to solve satisfactorily * * *" is a very pertinent observation. The importance of this factor in present-day construction is well illustrated by the fact that the cost usually charged against horizontal joint treatment, commonly termed joint cleanup, is still an appreciable percentage of the total cost of mass concrete in place. At Grand Coulee Dam, where the final sandblast method of cleanup is being used, the actual net cost of cleanup for 1,312,941 cu yd of concrete placed during the first half of 1939 was about 23 cents per cu yd. This cost was obtained through the courtesy of Edgar F. Kaiser, Assoc. M. Am. Soc. C. E., project manager for Consolidated Builders, Inc., contractors on the completion of Grand Coulee Dam, who were able to reduce the sandblast cleanup cost to the reported 23 cents from a considerably higher value, only after extensive research to develop suitable equipment and operating technique. Less than one third of this cost fully covers the sandblasting operation and its maintenance. The remaining two thirds represents the cost of removing construction wastes, mucking, final wash-down and maintenance incidental to these operations which are common to all thorough cleanup procedures under present-day methods of construction. It is apparent that the development of joint cleanup methods which will insure satisfactory results more economically presents a promising field for investigation.

Mr. Steele has indicated certain precautions to be observed and methods to be followed in order to "solve satisfactorily" the joint cleanup problem, but it appears desirable to define what can be considered as satisfactory results. Certainly there is no object in striving for bond strength and watertightness at the horizontal construction joints in excess of the tensile strength and impermeability of the concrete mass. From a study of Fig. 26, which summarizes the results of very carefully conducted laboratory tests on construction joint bond tests, it appears that 100% bond strength, or more, can be obtained by any one of several methods of treatment. However, these results should be interpreted in terms of field conditions, which require a margin of safety and a human-element-proof set-up to provide for the uncertainties of construction variables, and not in terms of the ideal laboratory control under which the tests were made.

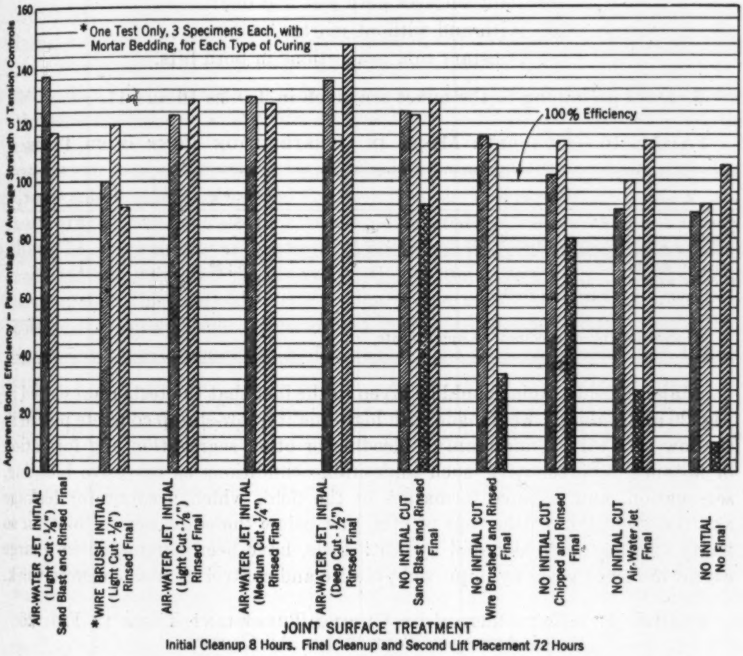

FIG. 26.—RELATIVE BOND EFFICIENCIES FOR VARIOUS COMBINATIONS OF CLEANUP AND CURING METHODS (AVERAGE RESULTS)

In appraising these test data, the following items apply:

1. The bond strength was measured in direct tension;

2. Efficiency was based on the average strength of solid tension control specimens;

3. The test specimens were 8-in. by 16-in. cylinders with mid-sections reduced to a 6-in. diameter;
4. At the time of the test the age of the bond was 28 days; and
5. The data represent the average results of 81 separate tests involving:
    - (*a*) Ten combinations of cleanup;
    - (*b*) Four types of curing treatment;
    - (*c*) Joint surfaces both wet and dry;
    - (*d*) With and without mortar bedding;
    - (*e*) Constant mix proportions in both lifts.
6. Data pertaining to the mixes are given in Tables 16 and 17.

TABLE 16.—CONCRETE MIXES IN CONSTRUCTION JOINT BOND TESTS

Description	Concrete	Bedding mortar
Cement content[a] (barrels per cubic yard)	1.18 ±	2.12
Water-cement ratio (net) by weight	0.60	0.60
Average slump (inches)	2	9
Mix, by weight	1 : 3 : 5[b]	1 : 3
G/S, by weight	1.6	....
Maximum size of aggregate (inches)	1.5	....

[a] Laboratory blend, modified. [b] Approximate.

Major consideration should be given to the fact that the tests represented in Fig. 26 and Table 17 were made with high-quality, low-slump concrete placed in shallow lifts with a minimum of bleeding or other segregation and formation of laitance. Fortunately, such undesirable conditions as excessive bleeding, segregation, and laitance formation in the field, which account for leakage and disintegration at the tops of lifts, just below the construction joint, in so many existing dams as noted by Mr. Steele, have been eliminated to a large extent in recent years by improved practice and control in mass concrete work.

TABLE 17.—EXPLANATION OF CURING TREATMENT TESTS IN FIG. 26 (Averages, Tops Wet in All Cases[a])

Item No.	Symbols (see Fig. 26)	Air temperature (F)	Humidity	Each increment value determined by:	Mortar bedding
1	▩	70°	50%	4 tests of 3 specimens each	With and without[a]
2	▨	....	....	2 tests of 3 specimens each[b]	With and without
3	▦	....	....	2 tests of 3 specimens each[c]	With and without
4	▨	....	....	1 test of 3 specimens[d]	With mortar bed only

[a] Item 1, also with tops dry. [b] Continuous spray. [c] Clear curing compound. [d] Two-inch layer of moist sand.

In addition to the tests represented by Fig. 26 and Table 17, many tests have been conducted in the field at various dams by casting columns on the surfaces of concrete lifts for either pull-off or push-over tests. The results of these field tests, together with observations on the jobs, are considered essential to the proper interpretation of the laboratory results illustrated.

The method of curing the surfaces of concrete lifts with a membrane curing compound was included in the laboratory tests merely for purposes of comparison. As might be expected, the results clearly indicate the difficulty in completely removing the membrane compound, which is obviously necessary if complete bonding is to be obtained.

Although the results shown in Fig. 26 indicate that satisfactory results (bond strength equal to 100% of the tensile strength of the concrete), under laboratory conditions, can be obtained by wire-brushed cleanup, either as an initial or final operation, the improvement in bond obtained by methods other than wire brushing is apparent. Such improvement is much more pronounced under field conditions where it is humanly impractical to secure effective and thorough cleaning of a large lift surface by wire brushing. Furthermore, the method is expensive.

Fig. 26 and Table 17 show conclusively the adequacy, under the conditions represented, of the air-water-jet method of initial cleanup, a procedure which has been popular in large dam construction during recent years. However, there are several factors on an actual job which necessitate caution and constant vigilance if comparable results under field conditions are to be secured with this method. The optimum age of the concrete at which the cutting operation should be performed varies considerably (from 3 to 12 hr), depending upon temperature and humidity, the type of cement, and the intensity of the air-water jet employed. If performed too early or with a jet under too high a pressure, permanent disruption of aggregate particles and overcutting with unnecessary waste of concrete are probable; if too late, the removal of laitance is not always insured. Proper bonding at a deeply cut surface (which is the usual practice on the job) requires particular care and thoroughness in working the mortar layer and new concrete into the old surface, an operation which is subject to human-element variations. Under many job conditions, the surfaces of the lifts become contaminated with construction wastes and coatings of various kinds before the next lift of concrete is placed. Such conditions require sandblasting or other suitable treatment in a final cleanup operation. On a large modern dam, it is reported that contamination subsequent to the initial cleanup is prevented by frequent washing with the high-pressure air-water jet during the interval of time that the lift is exposed. Complete cost data on this method are not available, but it is known that the cost of initial cutting and subsequent jetting on this dam is more than the cost of the actual sandblasting operation, exclusive of the cost of handling the sand, used in a final cleanup method on another job. Less than 6 cu yd of sand is required for cleaning a 50-ft by 50-ft block surface.

The sandblast method of final clean-up, just prior to placing the next lift, eliminates most of the uncertainties imposed by the aforementioned construction conditions, and thus goes a long way toward providing the needed factor of safety for insuring satisfactory results. Considerable savings in construction costs have also been effected on some jobs by the use of the sandblast final cleanup as compared with an initial air-water-jet method, followed by necessary final cleaning to remove contaminating coatings.

The results shown in Fig. 26 and Table 17 indicate that exposure of the lift surfaces to laboratory air at 70°F and 50% relative humidity has little effect on bond strength as compared with moist curing. This should not be interpreted as indicating that adequate curing is not required under field exposure to hot, dry air and sun. Proper curing of lift surfaces is considered essential under job conditions.

Of particular interest are the consistently superior results obtained in the laboratory tests with the damp sand-blanket method of curing. In these tests, the sand cover was applied eight hours after placing the first lift and, therefore, was not as effective in insuring a tight bond as would have obtained had the sand been applied at two to four hours, which is indicated by supplemental tests and field experiences. The damp sand method of curing, properly applied as soon after a lift is completed as practicable, constitutes excellent insurance against many of the unfavorable conditions for satisfactory joint bonding occurring on the job by preventing the formation of laitance and calcium carbonate and by protecting the concrete surfaces from injury, contamination, and drying. In order to effect these benefits, concrete of fairly stiff consistency (−2-in. slump) must be used and the sand covering applied at a very early age of the concrete, preferably before initial set takes place. With sand curing of concrete, which is reasonably free of bleeding and segregation, and is properly placed and compacted without overworking, and with the lift surface otherwise left in a suitable condition, about all that should be required before placing the next lift is removal of the sand and thorough cleaning with air and water jets. This method is being used on a large dam in which concrete placing has just begun. The early experiences, which may not hold under all the conditions that will be encountered throughout the life of this job, indicate that satisfactory results are being obtained at a very reasonable cost as compared with other methods. Sandblasting can always be used in conjunction with sand-blanket curing in spots where air-water washing might be inadequate.

The final step in any procedure for obtaining a good construction joint is concerned with placing the new concrete on the cleaned surface of the preceding lift. A common practice is to cover the surface with a half-inch layer of mortar in which the new concrete may be embedded. It has been demonstrated that good bond can be obtained by placing the new concrete directly on the old if the new concrete is worked very thoroughly. However, under the variable conditions and rush of construction operations, the insurance of complete bonding and contact afforded by the mortar layer is considered a very important factor and therefore its use is essential.

R. F. Blanks,[93] M. Am. Soc. C. E. (by letter).—Mr. Tyler has considered his subject of concrete control from its broadest interpretation, and has ably discussed factors in connection with the selection, production, and combination of materials and methods of construction which affect structural design conditions as well as the service performance of the finished structure.

Type of Cement.—He compares normal, modified, and low-heat cement for use in concrete dams. Normal cement is apparently included for the purpose of comparison as to physical and chemical properties. However, in the light of present-day knowledge of portland cements, the definite statement is warranted that normal cement should not be used in the construction of large masonry dams. This does not mean that reasonably satisfactory results cannot be obtained with normal (standard) cement provided construction rates are sufficiently slow and other important factors are appropriately controlled. However, modified, low-heat, or, under certain conditions, portland-puzzolan cement are so superior for masonry dams constructed with modern methods that there is no reason or justification for using standard cement.

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The principal difference between standard and modified cement, if they are both ground to the same degree of fineness, is that the latter contains considerably lower percentages of tricalcium aluminate ($3CaO \cdot Al_2O_3$) and somewhat lower percentages of tricalcium silicate ($3CaO \cdot SiO_2$). Tricalcium aluminate has been shown, by extensive tests, to be an undesirable constituent of portland cement for use in hydraulic structures, and particularly so if corrosive waters are involved. Concrete made with cement containing high percentages of $3CaO \cdot Al_2O_3$ are characterized by greater volume changes, lower ultimate strengths, and reduced resistance to disintegrating agencies, than concrete made with cement containing lower quantities of this constituent. Lower $3CaO \cdot Al_2O_3$ and reduced $3CaO \cdot SiO_2$ appreciably reduce the rate of heat generated by hydration, particularly at early ages. Thus, low-heat cement in combination with slow construction, or in combination with artificial cooling and rapid construction, will result in the setting heat being dissipated at a rate more nearly equal to the rate of heat generation, with attendant lower temperature differentials between interior and surface concrete, and with reduced cracking tendencies.

The use of low-heat cement without any provision for controlling concrete temperatures during construction will not necessarily eliminate cracking entirely. The flow of heat in concrete is susceptible of accurate analysis, and a decision to use low-heat in preference to modified cement should not be made until such analyses have been completed, taking into account anticipated construction conditions and practical methods of temperature control. It has been demonstrated that, with a reasonable spacing of contraction joints, large dams can be built with practically no temperature cracking. Reduction in cracking, together with elimination of joints, involves the question of economics. As improved methods of construction are devised and advances made in the development and use of materials, it is entirely conceivable that concrete dams may be built without joints and with no temperature cracking.

Materials and Concrete Production.—From Mr. Tyler's comments on concrete control, one cannot avoid being impressed by the fact that the production of concrete for a large dam under modern, rapid rates of construction is essentially a large-scale, intensive manufacturing process. Typical of such processes, the uniformity of the manufactured product, and in the case of masonry dams, the integrity of the finished structure, is dependent to a major degree upon the uniformity of the ingredient materials and the efficiency and control with which they are combined.

With the modern, efficient equipment that is available, raw aggregate materials can now be processed extensively for large jobs very economically. In large dam construction, therefore, there is little excuse for the use of aggregates which are not of satisfactory quality, grading, and uniformity, even with relatively poor raw materials. For example, uniformity of sand grading on current large jobs is easily maintained such that the variations in fineness modulus from the average do not exceed ±0.10. Such characteristics in aggregates pay large dividends in increased cement economy and improved handling qualities of the fresh concrete, with greater economy in construction and maintenance.

Progress in the production of uniform cement has not been so satisfactory, particularly when the cement supply originates from several mills, as is usually the case for large jobs. It has not as yet been found practicable to devise cement specifications that will insure any high degree of uniformity between different brands of the same type of cement. It is not uncommon to find two brands of cement supplied under the same specifications varying as much as 100% in 28-day strength and differing materially as to heat generation, water requirement, and other mass concrete-making properties. Such conditions require extensive blending facilities on the job and close scheduling of cement shipments from the various mills in order to obtain a uniform product.

Modern batching plants have been referred to as "houses of magic." This is an appropriate designation, as the efficient manner in which up-to-date equipment proportions the ingredient materials for mass concrete with a minimum of the human element and other variable factors, and a resulting maximum degree of uniformity, is in reality little short of magic. Developments in mixing equipment have not progressed in step with batching methods. In fact, until quite recently, equipment obtainable for mixing mass concrete containing large sizes of aggregate produced an action more nearly comparable to a ball mill than a mixer. Recently devised methods for measuring the efficiency and uniformity of mixing concrete have effected considerable improvement.

Construction Methods.—Recent advances in knowledge concerning the properties of mass concrete resulting from contemporary engineering research, as mentioned by Mr. Tyler, are gradually narrowing the gap between the designing office and the construction field.

The outstanding factor which distinguishes mass concrete from other types of concrete work is the development of cracking caused by temperature rise from the heat of hydration and subsequent cooling. To insure stress distribution and transfer in accordance with monolithic design the control of such cracking introduces problems of temperature control, foundation treatment, and limitations in construction procedure. These problems assume magnified proportions under modern, rapid-rate construction methods. It is safe to state that practically all of the unusual and varied control problems in connection with present-day mass concrete construction have been introduced as a result of speeding up the operations. For example, there were comparatively few mass-concrete problems, as engineers of today know them, involved in the construction of Crystal Spring Dam, in California, built about 50 years ago at a very slow rate. In this connection, Mr. Tyler's comments under the heading of "Concrete Control: Cracking" are particularly timely.

Construction Control.—Although little could be written concerning the details of construction control operations in a paper of this scope, Mr. Tyler's comments clearly reflect the complexity of the problems involved and the numerous items that require constant attention throughout the aggregate production, batching and mixing, transportation, placing, curing, and temperature control phases of modern concrete dam building. This, in turn, necessitates a large inspection force and adequate laboratory facilities with resulting relatively high engineering inspection and overhead costs. This mounting cost of engineering control, in terms of a percentage of the total cost, has been a source of considerable concern to engineers charged with the responsibility of building large projects. However, it should be borne in mind that many of these large projects would be economically unfeasible without the modern methods of construction. Thus, although the overhead and engineering costs are relatively high, the total costs are thereby brought within practicable economic limits, and, at the same time, a longer, more serviceable life is insured.

Concrete-Placing Buckets.—Mr. Tyler mentions some factors to be considered in connection with the use of buckets of large capacity. The large bucket is one of the many construction developments brought about by the ever-increasing demand for greater speed and, presumably, increased economy. The economy resulting from the use of large buckets is counterbalanced by either one or both of two factors: (1) Increased cement content to provide a higher slump for a given water-cement ratio; or (2) increased cost of vibration and other consolidating operations. If the concrete from straight-side buckets of large size could be properly distributed as it is discharged rather than being dumped all in one pile, the objectionable features would be minimized. Francis T. Crowe, M. Am. Soc. C. E., has developed an 8-cu yd bucket at Shasta Dam, in California, which can be operated so that the discharge is effectively controlled and shows promise of reducing the objections to previous types of large-size buckets. Buckets of large capacity have proved to be of considerable benefit in very hot, arid locations where a rapid rate of concrete placing is necessary in order to maintain a live concrete working surface and thus avoid cold joints and unsatisfactory consolidation.

Construction-Joint Cleanup.—The writer must take issue with the statement (see heading, "Concrete Control: Cleanup, Curing, and Finishing") "that construction joints are probably the weakest locations in any concrete dam." The comments in the paper following this quotation substantiate the writer's belief that the weakest locations are in the concrete near the tops of the lifts and not in the joint itself. Many tests, conducted both in the field and laboratory using various test methods, have shown that where reasonable care is exercised in cleaning the joint surface, the bond obtained at the joint is usually stronger than the underlying concrete. This fact emphasizes the need for giving proper attention to the design of mass concrete mixes and to the placing and consolidating operations in order to avoid the undesirable conditions mentioned by Mr. Tyler. The foregoing comments indicate that a joint-treatment procedure which can be accomplished with the least cost and at the same time insure the desired results is the method that should be used. Sandblasting and washing just prior to placing the next lift accomplishes such results. Very recent experience indicates that much, if not all, of the sandblasting can be eliminated when damp sand covering is used for curing, provided that the sand is applied soon enough after finishing the lift, and that the character of the surface concrete after placing is satisfactory.

Cold-Weather Protection.—Mr. Tyler's statement (see heading, "Concrete Control: Cleanup, Curing, and Finishing") that "provision for protection against freezing for two weeks after placing may be necessary in some cases" needs some explanation. Test data as well as experience on the construction of dams in cold climates have shown that concrete is not injured by freezing after, say, three days of curing at a temperature of 50° F, provided opportunity is had at some later date for normal resumption of the hydration process. The degree of protection required depends upon the location of the concrete in the structure and upon the particular job conditions. "Hard and fast" rules applicable to all combinations of materials and conditions cannot be formulated.

Berlen C. Moneymaker,[94] Assoc. M. Am. Soc. C. E. (by letter).—Mr. Crosby presents an interesting discussion of the more common geologic and foundation problems of masonry dams founded upon rock, and illustrates these problems by specific reference to successful dams and dam failures on various types of rock and geologic structure. Although these problems are discussed more or less fully in numerous excellent papers by Charles P. Berkey,[95] M. Am. Soc. C. E., Kirk Bryan,[96] Chester K. Wentworth,[97] L. C. Glenn,[98] Warren J. Mead,[99] Affiliate Am. Soc. C. E., and other geologists, Mr. Crosby's treatment of them will be of considerable interest to engineers.

The thesis of Mr. Crosby's paper—that essential foundation conditions can be determined in advance of construction, that safe and successful dams can be built at most sites, and that dam failures are not entirely unavoidable—is now rather generally accepted by both engineers and geologists. However, the geologist must bear in mind that geology is not the only consideration involved in the selection of a dam site. In programs requiring the integration of several projects, the location of a given dam may be fixed within a very short stretch of the river which does not afford a wholly satisfactory dam site. In such cases, it is the duty of the geologist to locate the best site available, under the restrictions imposed, and to discover and to acquaint the engineers with all the geologic conditions which must be met by design or foundation treatment, or both.[100]

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[94] Senior Geologist, TVA, Chattanooga, Tenn.

[95] "Responsibility of the Geologist in Engineering Projects," by Charles P. Berkey, *Technical Publication No. 215*, A.I.M.E., New York, 1929, pp. 4–9.

[96] "Problems Involved in the Geologic Examination of Sites for Dams," by Kirk Bryan, *loc. cit.*, pp. 10–18; also "Geology of Reservoir and Dam Sites," by Kirk Bryan, *Water Supply Paper No. 597*, U. S. Geological Survey, 1929, pp. 1–38.

[97] "Geology of Dam Sites." by Chester K. Wentworth, *Technical Publication No. 25*, A.I.M.E., New York, 1929, pp. 78–96.

[98] "Geology of Dams and Reservoirs," by L. C. Glenn, *Technical Publication No. 215*, A.I.M.E., New York, 1929, pp. 97–110; also "Geology Applied to Dams and Reservoirs," by L. C. Glenn, *Proceedings*, Engrs. Assoc. of the South, Vol. 26, 1915.

[99] "Engineering Geology of Dam Sites," by Warren J. Mead, 2d Cong. on Large Dams, Question VI, Washington, D. C., 1936; also *Civil Engineering*, May, 1937, pp. 331–334, and June, 1937, pp. 392–395.

[100] "Bad Rock Limits TVA Dam Location," *Engineering News-Record*, October 21, 1937, p. 665.

Perhaps the outstanding feature of Mr. Crosby's paper is his outline of the stages and objectives of a complete geologic investigation of a dam site. The geologic investigation of dam sites by the Tennessee Valley Authority (TVA) is conducted along lines almost identical with those outlined.

The geologic work of the TVA is done by the Authority's own highly specialized staff, and reviewed periodically by geologists on the Authority's board of consultants. When a dam is seriously considered for a certain stretch of the river, a geologic field party is detailed to the locality. This party consists of a well-trained engineering geologist with considerable experience, and his assistant, also well trained in geology but with less experience. The geologic work done at a TVA dam falls into the following stages: (1) Preliminary investigation, (2) exploration, and (3) construction.

(*1*) *Preliminary Investigation.*—This investigation embraces more than a brief reconnaissance study. It includes studies of the physiography, stratigraphy, and geologic structure of the area involved, as well as studies of the character and thickness of the overburden. In the course of this work, a geologic map of the area, showing in detail the various types of rock, contacts, faults, and joint patterns, is prepared on a scale of about 1 in. to 500 ft. Throughout this stage, very close cooperation is maintained between the geologist and engineer, so that the site finally selected for exploration is the one that best meets all requirements.

(*2*) *Exploration.*—The geologic work in the exploration stage involves the development of the details of structure, lithology, and rock conditions. By means of drill holes, trenches, pits, and tunnels, the configuration of the top of rock is determined, the overburden is classified, and the extent of rock weathering and cavitation is determined. Numerous geologic sections are drawn to show the structure, lithologic units, cavities, and zones of rock decay in as much detail as possible along various ranges. Very close cooperation between the geologists and engineers is continued throughout the exploratory stage.

(*3*) *Construction.*—A geologic field party is maintained on each project throughout the construction period. In this stage, hundreds of large-scale drawings are made, showing in detail all the geologic features that have any bearing upon engineering problems. These drawings, supplemented by numerous daily discussions with the geologist, keep the engineers informed in advance of all conditions to be met. In no stage of a project is the geologist more useful than in the construction stage.

Although a geophysical party was maintained, for a while, by the Authority's Geologic Division, it has been found desirable to limit the work of the geologist to geology, leaving the work in the allied sciences to specialists in those fields. Thus, all work in soil mechanics is done by experts in this field; the actual testing of materials is left to materials testing experts, etc.

Mention is made by Mr. Crosby of the usefulness of petrographic studies of the rocks involved in dam sites. The writer has found this method of study of considerable value, especially in areas of volcanic rocks. Many of the basic igneous rocks—basalts, andesites, basic tuffs and agglomerates, and diabases—contain hydrous silicate minerals of the zeolite family as veinlets and cavity fillings. As the zeolites are unstable and undergo volume changes in response to changing conditions of humidity, they can effect the complete disintegration of the enclosing rock in a very short time. The appearance of rock containing these minerals is often deceptive. Rock that is hard and apparently sound may go to pieces in a few days after its exposure. Fortunately, however, a petrographic study will reveal in advance whether zeolites are present in amounts sufficiently large to be dangerous.

Another geologic feature that may affect a dam (especially a high dam) adversely is sheet jointing or "sheeting planes." Massive rocks of igneous or metamorphic origin are likely to be divided into sheets by joints developed parallel to the topographic surface. Although the origin of these structures is perhaps debatable, it is generally attributed to relief of stresses as a consequence of unloading by erosion. In the abutments at a dam site, the sheet joints are inclined toward the river; in the rock underlying the river they are horizontal or inclined slightly in a downstream direction. These structures are generally marked by zones or "seams" of permeable decayed rock and, unless they are removed from the foundation or sufficiently treated, may impair the success of the dam by facilitating leakage or by resulting in uplift or settlement. At Hiwassee Dam, lenses of rotten rock developed along sheet joints as thick as 3 ft alternated with thinner or thicker layers of sound rock to a depth of more than 50 ft below the bed of the river.

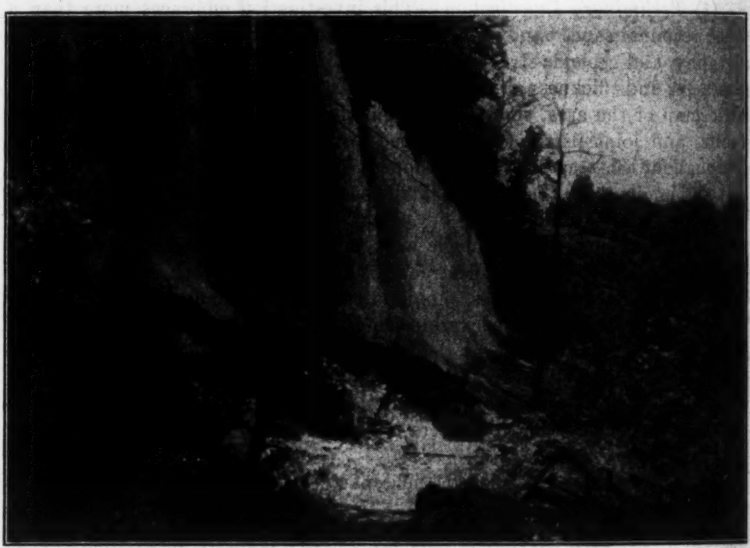

FIG. 27.—RIM LEAKAGE AT GREAT FALLS DAM, IN TENNESSEE

Mr. Crosby cites the enormous leakage at Hales Bar Dam on the Tennessee River as an example of what is likely to happen at a dam built without a geologic investigation. That a geologic investigation does not necessarily insure the success of a dam is strikingly attested by another masonry dam on flat-lying and cavernous limestone in Tennessee. The Great Falls (Rock Island) Dam on Caney Fork River had the advantage of geologic investigations by three reputable consulting geologists of very broad experience. Reports furnished the TVA show that these gentlemen recommended the site as suitable for a dam 110 ft high. The dam was built to a height of 35 ft in 1916, and in 1925 it was raised to 60 ft—50 ft lower than the height recommended. After the dam was raised, leakage appeared through the abutments and through the narrow rim that separates an arm of the reservoir from the river gorge below the dam. As the water short-circuiting the dam washed out clay seams and cavity fillings, an increasing quantity of leakage developed through the reservoir rim, producing a series of spectacular waterfalls downstream from the dam (see Fig. 27). A much greater portion of the flow of the river at this project is lost through leakage than is the case at Hales Bar.

Although other instances like Great Falls could be mentioned, there can be no doubt that the geologist of today is contributing more to the success and safety of dams than the geologist of a decade or so ago. Moreover, the geologist of today, who remains at a project from the initial exploration through the construction stage, contributes materially to the saving of money on foundation preparation and foundation treatment.

A. Warren Simonds,[101] Assoc. M. Am. Soc. C. E. (by letter).—In connection with the design of contraction joints, Mr. Steele has discussed the spacing of the joints, types of keyways, and the metal sealing strips for holding the fluid grout within the joint until the cement has set. In the construction of contraction joints, a good design may become of questionable value due to faulty construction at the metal seals or by the plugging of essential header pipes. Only too often do contractors consider the items used in contraction-joint construction to be merely pay items which, after being installed, are of no further value. This is usually true where the placement of large quantities of concrete is involved. Rigid inspection in such cases is essential.

In grouting vertical radial joints in a high arch dam in a narrow canyon, it becomes necessary to do the grouting in lifts. In such cases it has been observed that the grouting of every lift tends to open a V-shaped opening in the lift below, thereby splitting the bond of the grout film and preventing the blocks of the dam from having a complete bearing. In an attempt to remedy this difficulty a 3-in. horizontal cutoff groove has been tried in place of the horizontal sealing strips. This would permit the regrouting of a lift while grouting the next lift above. From the standpoint of the grouting operations, the cutoff groove was not as satisfactory as the sealing strip because the grout tended to run by the groove and foul the grouting system of the next lift above. Where artificial cooling of the concrete is used, present practice is to use horizontal sealing strips and to subcool the concrete prior to grouting. As the joints are grouted at the time when the temperature of the concrete has reached the minimum, the subsequent expansion of the concrete due to rising temperature would tend to close the open spaces which may possibly exist within the upper part of the joint.

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[101] Engr., U. S. Bureau of Reclamation, Denver, Colo.

In designing the grout system for an arch dam, the height of the grouting lift should be governed by the stability of the blocks when the joints are subject to grout pressure. The upstream component of the grout pressure in radial joints may cause tensile cantilever stresses of considerable magnitude at the downstream face of the blocks. The usual heights of grouting lifts range from 50 to 100 ft where grout pressures of 25 to 50 lb per sq in. are used at the top of the lifts.

The ideal joint grouting job would be one which permits the simultaneous grouting of all joints in the dam for their full height, as stated by Mr. Steele. As this procedure is rarely possible, due to the dimensions of most dams, the logical procedure is to grout all the joints up to the same elevation at one time. This can be done by connecting the headers of the joint grouting systems to a common supply line from the grout pump. Known quantities of grout can then be injected into each joint in rotation, thereby maintaining the surface of the grout at approximately the same elevation in the joints. This tends to reduce the unbalanced tipping of the blocks in an axial direction to a minimum. Filling one joint at a time is likely to cause an excessive opening of the joint being grouted, while the adjacent joints may be squeezed shut.

In grouting the joints of a large dam, it is not feasible to grout an entire lift of the dam from abutment to abutment in one operation. Groups consisting of 12 to 36 joints are therefore grouted as units. The grout pressure in the boundary joints is balanced by holding water under pressure in the adjacent joints. This pressure is regulated so that the closing of the adjacent joints is controlled until after the grout is set.

Control of the grouting operations should be governed by careful observation for the deflection of the blocks while grouting. Dial gages registering to 0.0001 in. are valuable for mounting across the contraction joints to observe the spreading of the joints. Whitimore type strain gages can also be used for this same purpose. Deflection at the top of the blocks can be observed with a theodolite. The magnitude of the deflection of the blocks of a dam depends on the dimensions of the blocks. In general, the opening of a contraction joint due to grout pressure should not be greater than 0.02 in. at the top of a lift having a height of 50 ft. Undesirable tensile stresses may develop if greater spreading occurs. Upstream radial deflections of the blocks varying from ¼ to ⅜ in. have been observed while grouting the joints of arch dams having heights of approximately 200 ft.

The number of joints that can be grouted in one operation depends on the equipment and organization available. A total joint area of 100,000 sq ft (neglecting keyways) can be grouted in an 8-hr shift using one 10 by 3 by 10-in. duplex grout pump and one grout mixer having a capacity of about 20 cu ft. Slow grouting is desirable in injecting the grout. The length of time required to fill one large joint may be as much as twelve hours. Where the temperature of the concrete is low, the grout will remain fluid longer than when the temperature is high.

The properties of cements used in contraction joint grouting vary widely. Standard portland cement, high early strength cement, modified cement, and slow setting oil well cement have been used. Cements purchased under the same specifications vary widely when supplied from different mills. In general, an air-separated cement is to be preferred, although an extremely fine-ground, air-separated cement may become lumpy before use, due to "warehouse set." The mechanical screening of cement at the job will produce the best cement for contraction joint grouting although its cost will be relatively high. Such a cement can be delivered to the mixer free from lumps, unground clinker, and tramp iron. A good cement for contraction joint grouting should have a fineness such that 100% will pass the 100-mesh U. S. standard screen and 95% will pass a 200-mesh screen.

A contraction joint grouting system is a fairly costly feature of dam construction. For best results, care should be used to effect an adequate job of grouting. The joints should be thoroughly cleaned by washing with water and compressed air. The amount of muck from construction that may accumulate in a contraction joint is often surprizing in quantity. After washing, the joints should be soaked with water for twenty-four hours immediately prior to injecting the grout. The consistency of the grout used varies from 2 parts of water to 1 of cement by volume to 0.6 part water to 1.0 of cement in extreme cases. The usual mix ranges from 1 : 1 to 0.7 : 1 of water to cement.

Reliable and suitable equipment is essential in joint grouting. In order to reduce the hazard of equipment failure, it is desirable to provide a standby unit which can be put in operation in the event of an emergency. An extra grout pump should be considered a standard part of the equipment. Lubricated plug valves on the supply headers have proved the most satisfactory for use with grout under pressure. Use of this type of valve reduces the sticking and binding so often experienced with cheap valves.

This Symposium presents the most recent assumptions and theories involved in the design of high and important masonry dams of the single-arch, curved gravity and straight gravity types. Much of this information has been obtained from experience in the design and construction of some of the world's largest dams. The paper by Mr. Steele is a necessary part of the Symposium as it covers the treatment of construction joints in mass concrete. This paper contains some much needed information for engineers engaged in the design of concrete dams and also tables of data which serve as a record of the design and method of treating construction joints.

W. J. E. BINNIE,[102] HON. M. AM. SOC. C. E. (by letter).—Messrs. Houk and Keener are to be congratulated on the lucid manner in which they have dealt with their subject, but they make certain assumptions for the purpose of computation which the writer considers are open to question. Assumption 3 deals with the foundation rock which must be assumed to be homogeneous and uniformly elastic in all directions, a condition which Messrs. Houk and Keener consider is somewhat open to question. The writer has had some 50 years of experience with foundation rock and has seldom had the good fortune to secure absolutely undisturbed and homogeneous material.

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[102] Civ. Engr. (Binnie, Deacon & Gourley), Westminster, S. W. 1, London, England.

Assumptions 8 and 9 deal with the modulus of elasticity, which is very difficult to estimate with accuracy for the following reasons: Many investigations have shown that concrete does not behave as a truly elastic material, the deformation under any given load being dependent more on "flow" or "creep" than elasticity. Messrs. Houk and Keener express the opinion that this characteristic can be allowed for adequately by the adoption of a somewhat lower "modulus of elasticity" than would otherwise be adopted for use in a technical analysis. Now, the modification that must be made is large and depends upon many considerations, to which the writer proposes to refer briefly.

It is only in comparatively recent years that the factors that govern the deformation of concrete under any given load have been studied, and much is still to be learned. Most information available in England is based on the experiments of R. D. Davies,[103] Oscar Faber,[104] and the work done at the Building Research Station, Watford, England, under W. H. Glanville.[105]

It has been shown: (1) That the deformation caused by the flow of concrete under sustained load may far exceed that due to the elastic modulus; (2) that this deformation depends upon the age of the concrete when loaded; (3) that it continues over a long period, reaching ultimately a "limiting" value; (4) that it is dependent upon the cement used; (5) that it is dependent upon the strength of the concrete mix; (6) that the degree of saturation with water has a marked effect in decreasing the "flow" of concrete that has been saturated with water; (7) that the grading of the aggregate also has a considerable influence; and (8) that computations should take account of changes in the volume of concrete caused by varying degrees of saturation, which also affect the coefficient of expansion and contraction due to changes of temperature.

The "flow" of concrete made with rapid hardening, or alumina, cements (see item (4)) is less than half that which occurs when normal portland cement is used, unless the concrete is not immersed in water, in which case the "flow" under any given load is greater for concrete made with rapid hardening and alumina cements.

The data in Table 18 are given by Glanville as an illustration of the effect on the modulus of elasticity due to varying the mix (item (5)), the concrete being made with normal portland cement loaded after twenty-eight days and kept in an atmosphere of uniform humidity.[106] The term "limiting" value (item (3)) denotes the modulus attained when "flow" ceases.

When it is complete, a dam will be subject to variation in the degree of saturation of the concrete (item (8)); that is, the portion above water level may become comparatively dry due to exposure on both faces to a hot sun, whereas the submerged portion will be partly saturated.

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[103] "Flow of Concrete Under Sustained Compressive Stresses," by R. D. Davies, *Proceedings*, Am. Concrete Inst., 1928, pp. 303–335.

[104] "Plastic Yield, Shrinkage and Other Problems of Concrete and Their Effect on Design," by Oscar Faber, *Proceedings*, Inst. of Civ. Engrs., 1927–1928, Pt. I, pp. 27–130.

[105] "The Creep or Flow of Concrete Under Load," by W. H. Glanville, *Building Research Technical Paper No. 12.*

[106] *Structural Engineer*, February, 1933, p. 57.

Moreover, the age of the concrete where the stress due to water level is imposed can scarcely be the same throughout the entire structure. The "effective modulus" for the lower part of a high dam probably will have nearly reached the "limiting value," whereas the age of the highest part may be only a year by the time the reservoir is filled, and the "effective modulus" applicable in the two cases may differ by a considerable amount.

TABLE 18.—EFFECT OF MIX PROPORTIONING ON THE MODULUS OF ELASTICITY

(All Units, Pounds Per Square Inch)

Mix by weight	Modulus of elasticity	EFFECTIVE MODULUS DUE TO INFLUENCE OF FLOW	
		At twelve months	Limiting value
1 : 1 : 2	$5\ 3 \times 10^6$	1.7×10^6	1.4×10^6
1 : 2 : 4	3.4×10^6	0.76×10^6	0.63×10^6
1 : 3 : 6	2.9×10^6	0.47×10^6	0.40×10^6

Under "Stress Conditions," Messrs. Houk and Keener refer to Poisson's ratio effect, but experiments at the Building Research Station at Watford, on columns under compression, showed that they shrank laterally, when loaded, to about the same extent as they had when unloaded. As far as the writer is aware, little is known of the effect of varying degrees of saturation of concrete on changes of volume and their influence on the temperature coefficient, but Messrs. Houk and Keener, with their wide experience, will probably be able to cite some literature on the subject.

In making the foregoing remarks, it has not been the writer's intention to detract from the value of mathematical analysis, but to indicate that the foundations on which the mathematical structure rests are not so fully known as to justify too great a degree of refinement.

Under assumption 12 (heading "Load Conditions"), Messrs. Houk and Keener express the opinion that the silt contents of flood waters usually may be neglected in designing storage dams but may require consideration for relatively low ones. This opinion is not shared by all engineers. At the meetings of the Grand Barrage Section of the World Power Conference held at Tokyo, Japan, in 1929, no less an authority than the late Allen Hazen, M. Am. Soc. C. E., contributed a paper recommending that dams should be designed on the assumption that water weighed 100 lb per cu ft to allow for the possibility of silt pressure.

When constructing the Silent Valley Reservoir for the water supply of the Belfast and District Water Commissioners in North Ireland, the problem of sinking a trench to a great depth through "morainal" material saturated with water had to be faced; and, as much of it consisted of quartz ground to a powder, it was apprehended that the pressure on the sides of the trench would be very high, calling for the adoption of special methods to give the necessary support to the sides of the trench.

It was ascertained by experience that this finely ground saturated material behaved as a liquid weighing 100 lb per cu ft, and the use of timber had to be abandoned and cast iron substituted.[107] In the case of low dams impounding heavily silt-laden water, the writer would adopt a liquid pressure of 100 lb per cu ft for the design, but agrees that it is only in very exceptional cases that this assumption is necessary for dams of any considerable height.

Messrs. Houk and Keener indorse the conclusion that it is seldom necessary to apply a straight-line pressure distribution from reservoir head to tailwater level to more than two thirds the area of the bank when considering uplift pressure. It is now usual to provide for drainage of the base of the dam close to the face, leakage water being discharged into galleries that may be only a little above tailwater level. It would appear that, if the drainage arrangements are adequate, the uplift pressure when foundations rest on non-porous rock would be small.

If the foundations of the dam penetrate rock to any considerable depth, the concrete becomes well bonded to the rock, owing to the irregularities in the sides of the trench, and the effective weight tending to resist "uplift" pressure is not only that of the dam itself but also that of the adjacent rock. The writer has sloped foundations toward the water face to increase security against sliding and so that the foundations at the water face may penetrate the rock to a sufficient depth to provide against possible "uplift" pressure, the slopes adopted varying from 1 in 12, for good sound rock, to 1 in 5 in the case of greasy shale.

Preparation of Foundations.—The paper by Messrs. Paul and Jacobs covers a wider field than might be gathered from the title and is full of instructive matter. Borings of small diameter which can be driven rapidly are of great benefit in enabling the engineer to select the best site for the dam; but they are likely to be misleading, as stated by the authors.

Experience at Silent Valley Reservoir[107] illustrates how one may be deceived. Preliminary borings were sunk across the valley in the usual manner, and the drills appeared to enter rock at depths that did not exceed 65 ft. Hence, a contract was let on the assumption that there would be no great difficulty with regard to the foundations. The rock was overlaid by morainal material which consisted of very fine sand and silt, saturated with water, in which were embedded large boulders left by the retreat of a glacier or ice sheet. It was these boulders that had been encountered, and not solid rock. The latter actually lay at a maximum depth of 196 ft from the surface, or 180 ft below the level of saturation of the morainal material. So numerous were the boulders that the contractor found it impracticable to drive piles to any depth.

The site would have been abandoned if it had not meant the sacrifice of a large sum of money which had already been spent on ancillary works before the true state of affairs was ascertained; and it was decided, therefore, to proceed with the work if it were possible to do so. Ultimately, the dam was successfully completed by the use of compressed air and novel methods of construction, which have been described elsewhere.[107]

The writer has not used large diameter borings for foundation exploration, but fully indorses the views of Messrs. Paul and Jacobs with regard to their advantages. The best course of all is undoubtedly to remove the overburden, or to excavate an exploratory trench if it is possible to delay letting the main contract until such work is completed. The writer agrees that abrupt changes in height above foundations are to be avoided, as cracks so frequently form at such points, and he has abandoned steps in favor of a sloping foundation.

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[107] "The Construction of the Silent Valley Reservoir, Belfast Water Supply," by G. McIldowie, *Proceedings*, Inst. of Civ. Engrs., Vol. 239, p. 498.

Geological Problems of Dams.—The writer has read the paper by Mr. Crosby with great interest and considers it a masterly exposition of the subject, which should be in the hands of all engineers concerned with dam construction.

In England, geological conditions are generally unsuitable for the construction of masonry dams. Clays, soluble limestone, and chalk cover a large area, and most reservoirs are formed by means of embankments with a puddle core, material suitable for puddling generally being obtainable in proximity to the site.

Concrete Control.—Under the heading "Concrete Control: Cleanup, Curing, and Finishing," Mr. Tyler states that surface vibration of the concrete should be avoided because of the danger of bringing mortar to the surface and remixing it with accumulated water. The writer believes in using as little water as practicable when mixing concrete, and has found that more water is required for the successful operation of internal vibration than of surface vibration. He has obtained excellent results by spreading the "green" concrete in layers not more than 6 in. thick, the mix being so dry that moisture appears on the surface only after 2 or 3 min of vibration. The tool that is used is a pneumatic vibrator which has a mushroom-shaped head. It is not possible to use surface vibration in many instances, but a concrete dam affords an excellent opportunity.

In the sentence preceding "Conclusion," Mr. Tyler refers to the deflection of a 265-ft gravity dam ⅜ in. upstream, against a rise in water level of 150 ft, due to seasonal temperature effects. The writer has noticed similar deflections upstream with increased water load in a case where temperature could scarcely account for it—namely, the Gorge Dam completed in 1936 for the supply of Hong Kong and Kowloon, China.

The dam is a composite structure with a maximum height of 275 ft above the stream bed, the water face being of concrete varying in thickness from 5 ft to 125 ft at stream-bed level, and the concrete portion is backed with hand-packed rock fill varying in thickness from 45 ft at 3 ft below top water level to 450 ft at stream-bed level.

Instruments were provided to measure the deflection from the vertical of the concrete portion of the dam under different conditions of water load, one being placed 93 ft above stream-bed level, the other recording the deflection at the top of the dam with reference to the level at which the lower instrument was placed.

Observations, which have now extended over a period of 3½ years, have revealed the following facts, which are difficult to explain. The lower portion of the dam deflects toward the water as the reservoir level rises, whereas the upper portion deflects in the opposite direction, the extent of these movements being as follows: In 1937 the movement from the vertical toward the water at 93 ft above stream bed was 0.16 in. with an increase in water level of 95 ft, and 0.14 in. with an increase in water level of 75 ft in 1939. There was a partial failure of the monsoon in 1938, the water level fluctuating irregularly, and the movements were small. The deflection from the vertical in the opposite direction at the top of the dam was more marked, amounting to 0.78 in. in 1937 and 0.7 in. in 1939 for the aforementioned increases in water level. Now if these movements were due to temperature changes, it is difficult to understand why the deflection of the upper and lower portions of the dam should be in opposite directions.

Construction Joints.—Mr. Steele is to be congratulated on the mass of valuable information contained in his paper. It was formerly the practice in England to make special provision for air slaking of any free lime in the portland cement before use.

When constructing the Vyrnwy Dam, the cement was placed in a lofty shed and caused to fall through the air from story to story, being aerated until the rise of temperature in setting did not exceed 3° F in 60 min or 2° in 15 min.[108] Another method was adopted by the writer's firm for the construction of the Alwen Dam, where the cement was placed in bins, provision being made to blow air upward through the cement. Perforated pipes were laid on the floor of the bins for that purpose. It was specified that aeration should continue for three weeks and for a period of not less than eight hours each day.

No provision was made for "transverse" joints in either dam, and the Vyrnwy Dam was free from all but hair cracks; but two vertical cracks appeared in the Alwen Dam at the abrupt change of section due to steps in the foundations. One of these cracks was difficult to detect and no percolation is visible, but slight percolation takes place through the other.

The practice of slaking the portland cement is not now followed in England. Instead, it is finely ground, and transverse joints have become desirable.

Under "Joint Spacing," Mr. Steele refers to the longitudinal joint in the Assuan Dam. In 1928 the Egyptian Government appointed an International Technical Commission, consisting of the late Hugh L. Cooper, M. Am. Soc. C. E. (for the United States), H. E. Gruner, M. Am. Soc. C. E. (for Switzerland), and the writer (for Great Britain), to consider the feasibility of increasing the storage by raising the dam a second time, and, in consequence of the Commission's Report, the dam was heightened by 9 m (29.5 ft).

The original dam was faced with cyclopean granite masonry, the "hearting" consisting of stones that could be handled, occupying 35 to 40% of the mass. The interstices were filled with 2 : 1 portland sand-cement mortar at the water face and near the foundations and 4 : 1 elsewhere, no transverse joints being provided.[109]

When it was decided to heighten the dam for the first time, it also became necessary to increase its width, the design of this work being entrusted to the late Sir Benjamin Baker, Hon. M. Am. Soc. C. E., who adopted the following procedure: A space of 30 in. was left between the old and the new work and, as this space was inclined to the vertical, steel rods 1½ in. in diameter, spaced 1 m apart, were embedded in the old and penetrated the new work, the shuttering being removed when the concrete had set. After the new work had been completed, a period of two years was allowed to elapse so as to permit the equalization of temperature before the joint was filled with fine granite chips and grouted. No transverse joints were provided in the raised portion.[110]

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[108] "The Vyrnwy Works for the Water Supply of Liverpool," by G. F. Deacon, *Proceedings*, Inst. of Civ. Engrs., Vol. CXXVI, pp. 44 and 45.

[109] "Nile Reservoir, Aswan," by Sir Maurice Fitzmaurice, *Proceedings*, Inst. of Civ. Engrs., Vol. CLII.

The first problem with which the Committee was confronted was to determine whether the old and new work had been so well knitted together that the entire structure could be considered as homogeneous. The numerous sluice culverts which traversed the dam afforded the opportunity of internal inspection, and the junction between the grouted joint and the old and new work was cut back in the roof, sides, and floor of every culvert so as to permit an inspection by magnifying glass. The decision reached was that the dam could be considered as homogeneous, justifying calculations on that assumption. It was considered, however, that it would be too sanguine to assume that the entire mass would be homogeneous when the dam was heightened for the second time, and hence the additional support is given by massive buttresses that can expand and contract without stressing the main structure. Non-corrodible steel plates separate the buttresses from the body of the dam.

Mr. Steele refers to the transverse joints that were provided when the dam was raised for the second time, and these are spaced 23 ft apart, this distance being dictated by the position of the buttresses. He does not appear to favor metal stops to prevent percolation across the horizontal joints that occur between concrete which has already set and that which is superimposed; and, no doubt, if the greatest care is taken in preparing the surface of the old work in the manner that he describes, seepage should be avoided. However, the writer has so frequently observed seepage along these planes that metal stops were used in the case of the Gorge Dam already referred to.

An inclined and articulated reinforced-concrete diaphragm is provided at the water face, supported by buttresses spaced 12 ft 6 in. apart, projecting from the main concrete structure, which is referred to as the "thrust block." The diaphragm alone is relied on for watertightness and varies in thickness from 3 ft at the top to a maximum of 6 ft at a depth of 172 ft below overflow level. It is made of concrete containing 600 lb of cement per cu yd, in place. It is divided into panels by vertical transverse joints, spaced 25 ft apart, formed of bent copper plates 0.1 in. thick, immersed in a bitumen compound, the joints coinciding with the center of alternate buttresses. The concrete was placed in lifts of 20 ft, and longitudinal copper stops 0.1 in. thick, which varied in width from 2 ft to 1 ft, according to the water pressure, were built in at the top of each lift, projecting either 1 ft or 6 in. into the lift above.

It was found that the maximum rise of temperature above that of the atmosphere of the concrete was almost proportional to the quantity of cement used per unit volume, and amounted to 53° F as an average for concrete containing 600 lb of cement per cubic yard, in place, the average crushing strength being 350 tons per sq ft.

Considerable economy was possible by reducing the quantity of cement used for the thrust block to 300 lb per cu yd, the crushing strength being 190 tons per sq ft and the maximum temperature rise being only 24° F above the atmosphere when setting.

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[110] "Protection of Downstream Rock Surface," by Sir Murdoch Macdonald, *Proceedings*, Inst. of Civ. Engrs., Vol. CXCIV.

The diaphragm is separated from the buttresses by means of special bituminous sheeting 0.2 in. thick, so that slight differential movement can take place without imposing stress. Similar sheeting was used to form the joint between adjacent panels and is interposed below the panels to separate them from the face concrete.

The space between the buttresses and behind the diaphragm communicates with galleries in the thrust block so that inspections can be made to ascertain whether leakage is taking place. It is gratifying to be able to record that only an occasional drip can be heard, which in all probability is due to condensation of the moist atmosphere.

In the case of the Gorge Dam the stresses due to water pressure are taken partly by the thrust block and partly by the rock fill. This composite structure was adopted partly because of the cheapness of Chinese labor and the immediate proximity of the granite quarries, which made possible the placing of hand-packed rock fill at a remarkably low figure. However, concrete crushing at 190 tons per sq ft affords an ample margin of safety to meet the stresses set up in a straight gravity dam of conservative design.

The writer would recommend the adoption of an articulated watertight diaphragm of a high-grade concrete, transmitting the thrust to the main dam, as considerable economy would be secured by adopting a concrete mix of lower grade for the latter, and all difficulties with regard to contraction or temperature cracks which permit percolation would be avoided.

HOMER M. HADLEY,[111] ASSOC. M. AM. SOC. C. E. (by letter).—The papers of this Symposium constitute an admirable presentation of both theory of design and construction practice which have obtained during the greatest dam building period in history. Unprecedented in height, great size, and number of large structures, the dams of this time have received the most careful, thorough, and painstaking engineering thought and attention—to which fact they will give witness for centuries and millenniums to come.

For satisfactory performance the concrete (Mr. Tyler states) must possess strength, weight, durability, impermeability and "continuity [that is, freedom from cracks] in order that the structure may act according to assumptions of design." The other authors either directly or implicitly indorse this judgment which unquestionably is the widely held and practically unanimous opinion among engineers today. Nevertheless it would appear that continuity might well be accepted as satisfactory even if it were of relative rather than of absolute degree and that impermeability of structure might be obtained, advantageously and economically, in numerous cases by means other than those of present practice.

The enemy of continuity in concrete is the crack which generally has its origin in thermal and moisture-content changes. Because of internal shrinkage, that which was whole and continuous becomes divided into separate parts by fissions or cracks. The structural consequences of such cracking may be serious or not, depending upon the width of the crack, the shape, form and direction of the surfaces of cleavage, and upon the physical structure, aggregate particle size, and characteristics of the concrete itself. A crack, $\frac{1}{16}$ in. wide, developing on what approximates a geometrical plane surface in a mortar having $\frac{1}{4}$-in. maximum-size aggregate would produce vastly different effects on stress distribution than would the same $\frac{1}{16}$-in. crack developing with warped irregular surfaces in a concrete made with 6-in. or 8-in. maximum-size aggregate. In the first case there would be practically complete loss of shearing strength in the plane of the crack, a complete loss of tensile strength perpendicular to the crack and, until the crack closed again, a complete loss of compressive strength. In the second case there would be only a slight loss of shearing strength, and even perpendicular to the crack at any point, due to aggregate interlock, a certain amount of tensile and compressive strength would persist. Let it be emphasized here that cracks in mass concrete do not form plane surfaces; they bend and twist to form warped and highly irregular surfaces which themselves interlock independently and additionally to the aggregate interlock. If, then, with normal dam concrete, cracks are not so closely spaced as to destroy columnar action in the sections between them, it is difficult to see anything particularly grievous structurally to result from them. It is to be recognized, of course, that there are cracks and cracks, some of which may truly be of a serious nature; but there are also others which are so utterly trivial and insignificant as to warrant little or no "dither" about them or the incurring of expense to prevent their formation.

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[111] Regional Structural Engr., Portland Cement Assn., Seattle, Wash.

There is the possibility, of course, that cracks in dams may become filled with water under pressure. In the case of cracks perpendicular to the axis it is generally agreed that no harm would result if such a thing did happen, but water under high pressure in cracks parallel to the axis would produce lateral thrusts tending to disrupt the mass by splitting off successive laminæ from the downstream face. Although such action may be assumed, of course, a much more cheerful and probable assumption to make is that nothing of the kind will occur. Cracks parallel to the axis of the dam are intersected at regular intervals by cracks perpendicular to the axis, which latter vent to the atmosphere at the downstream face of the dam. Consequently when and if pressures do tend to build up they relieve themselves via the perpendicular cracks. A wide field of assumptions can be made regarding the extent, direction, and pattern of cracks that may develop. However, certain things are inherently more probable than others. Because the length of a dam is almost certainly greater than its thickness, cracks transverse to its axis are inherently more probable than cracks parallel to the axis despite the fact that escape of heat and moisture from the mass is outward from the surfaces and therefore transverse to the axis. Therefore, to assume cracks parallel to the axis without the simultaneous existence of cracks perpendicular to the axis is to assume a possibility, yet scarcely a probability.

That water pressures in cracks, if they do tend to develop, find relief without causing damage, and that this is not mere wishful thinking, is attested by definite evidence. Prior to the present dispensation of crackless concrete, numerous dams were built which incorporated the errors now painstakingly avoided. It is true that these older dams are not of the magnitude of those of the present day, but they are fair-sized scale models of present dams. Being built with high-heat cement, in large masses, with uncooled mixing water and aggregates, without artificial cooling, and without the careful foundation treatments now provided, they developed cracks perpendicular to their axes and likewise parallel to their axes. Opportunities for water under high pressure to penetrate these cracks have been quite as favorable as any to be found today. Despite these opportunities for trouble and disaster (if such indeed they are), not one of them that the writer has ever seen or read about has suffered in consequence. These old dams present many a warning against watery, sloppy concrete, against segregation, against careless construction joints, against concrete so porous or weak that it is unable to withstand weathering or ordinary abrasion. Nevertheless, as far as offering any evidence corroborating fears about harmful high water pressures developing in ordinary shrinkage cracks, that is the thing they signally fail to do.

In many places freezing occurs, and then the question of possible damage from the freezing of water in cracks arises. The writer has seen a considerable amount of frost damage in porous concrete. This has been characterized by general surface scaling with increased disruption at horizontal construction joints with their laitance. On the other hand, where the concrete has itself been of a quality to be classed as frost resistant, he does not recall having seen damage at cracks. A very excellent example of such contrasting behavior was seen several years ago in a box flume on the Owens Valley aqueduct of the City of Los Angeles, Calif. This elevated box flume was a reinforced structure built with standard portland cement. The approach conduit sections were built with the tufa cement (puzzolanic) generally used throughout the aqueduct.[113] In about twenty years of occasional freezing and thawing, leaking cracks in the sides of the approach sections had spalled considerably around the edges of the cracks and over adjoining surfaces, whereas leaks in the box-flume structure along the construction joint between the bottom slab and the side walls had spalled not at all. This is not an analogous case, of course, to the freezing of water in cracks parallel to the axis of a dam; but instances of damage from such a cause are, to say the least, not of common record.

The writer never saw St. Francis Dam either before or after the catastrophe, the cracks in which are frequently referred to; but he has a vivid recollection of being shown certain hard fragments of foundation material taken from the very place in the stream bed upon which only a few days before the St. Francis Dam had stood. The behavior of the fragments of this "bedrock" was this: Dropped into a glass of water, they emitted some tiny bubbles and quickly softened, lost their shape, and flattened into a thin layer of mud in the bottom of the glass! If such was the behavior of these fragments pried from the St. Francis "bedrock" it seems quite possible that water stored behind the dam might, slowly, have worked its way through softened material beneath the rudimentary cutoff wall and beneath the dam and at last have piped through—first a trickle, then a jet, then a bursting stream, and finally a wild uncontrollable torrential flood that tore out the dam and swept headlong to the sea. The fact that longitudinal cracks were found in that block of the dam remaining in place after the disaster may indicate what was the primary cause of the failure or what was a lesser cause. Foundation conditions being such as they were, however, it scarcely seems necessary to seek other causes. At least it is to be said that the block of the dam which remained in place remained despite the longitudinal cracks in it. Whatever the initial cause, it would be well if Mr. Crosby's description of the failure of St. Francis Dam were widely read (see heading "Dams on Sandstone and Conglomerate").

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[113] "Tufa Cement, as Manufactured and Used on the Los Angeles Aqueduct," by J. B. Lippincott, Hon. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXVI, December, 1913, p. 520.

The inspection gallery that parallels the upstream face of Pardee Dam is approached by a gallery extending through the dam from the downstream face. Athwart this approach gallery is a series of longitudinal cracks paralleling the axis of the dam. These are of varying inclination and change from parallelism to the downstream face near that face to practically a vertical direction near the upstream face. On a number of occasions the writer had heard these referred to in rather sober, serious tones. It was somewhat surprising, then, on visiting the dam and being shown these cracks, that considerable difficulty was experienced in locating them. The approximate position of the first crack was known to the guide but it seemed impossible to find it in the darkness with a flashlight. Finally, however, a blue keel mark was found on the wall and beside it a fine crack, possibly 1/64 in. wide, which in a quite irregular course encircled the gallery. A repetition of this process of search and discovery at 30-ft or 40-ft intervals located the entire family of cracks, all of which, as now remembered, were dry. Although their varying directions were of interest, the writer was at a loss to understand what there was about them to give them a rating higher than insignificant. If the rock abutments and foundations supporting the dam contained cracks and fissures no wider or more closely spaced than these cracks they would be acclaimed as of outstanding excellence even though the jointing lacked anything corresponding to aggregate interlock which characterizes the crack in concrete.

This same aggregate interlock across cracks, particularly across cracks of smaller breadth, is not without points of distinct superiority to the smooth formed surfaces of keyed block jointing even though the latter is grouted. Mr. Steele's comments, under "Slots Versus Grouted Joints Versus Open Joints," indicate that grouting is not, by the fact itself, automatically successful for sealing joints. With aggregate interlock and without separation of parts it is impossible to have movement or slippage or any consequential magnitude along the general plane of a crack without complete shearing of the projecting aggregate, shattering of the adjoining concrete, etc. Without such concomitant manifestations the reported cases of movements and offsets at cracks of necessity must be slippages on pour-planes, unkeyed vertical joints, or similarly formed and created separations resulting from construction processes.

The point of the preceding discussion is in nowise to disparage standard practise, but definitely to advance the idea that small cracks developing in concrete dams are not the desperately dangerous things which so much current practise and discussion, either by implication or direct statement, indicate them to be. If it has been assumed that there will be no crack, then assuredly a crack does violence and affront to the assumption; but the existence of a small crack does not mean that stresses cannot and will not be transmitted across it. Neither does it mean, from the recorded behavior of dams containing cracks, so far as the writer is aware, that seriously harmful consequences will ensue from them. In short, their hazard appears to be more an assumed one than an actual one. Therefore, although cracks are, in general, to be minimized and avoided, if small ones do occur they can be contemplated with equanimity. They may impair the "structural integrity" of the dam slightly but the actual damage and harm they cause is very slight indeed.

Present practise in the construction of gravity dams is to provide durability, impermeability, and continuity throughout the entire body of the dam—that is, the same high quality of concrete is used in all parts, block construction with its forms and keyways and water-stopping and grouting are used, lifts are made shallow with time intervals for heat radiation and with careful cleaning of surfaces, and the concrete is cooled by circulating water through embedded pipes. By these measures a uniform quality of concrete and a uniform high degree of durability and impermeability are provided throughout the entire structure. There can be no question as the the splendid results achieved by these carefully executed methods and processes, and to question the sufficiency either as to structure or foundations of any of the great modern dams built of proved materials is indeed to utter idle words.

Nevertheless, as Mr. Steele observes (latter part of the section on "Joint Spacing"), "All of these factors affect the cost of the finished concrete * * *." They do that, of and by themselves, and to the further extent that it is wholly impossible with these numerous and time-consuming processes for work to proceed at other than a corresponding rate. Any one who has had opportunity to observe a concrete gravity dam under construction and has followed this with observation of a rolled-fill earth dam under construction, cannot fail to be impressed by the restriction and complexity of processes of the former, by the ease and simplicity of the latter, and by the obvious contrasting costliness of the two operations.

Consideration over a period of time leads the writer to believe that with respect to lowered cost and also in adaptability to foundations that might appear questionable for the present standard dam there is a field of usefulness for what may be called a "concrete fill" dam; that is, a heavy gravity section constructed of lean concrete to be built from the bottom up in a single mass, layer upon layer, in the same manner that rolled-fill dams are built, with impermeability and durability provided at the surfaces of exposure. Essentially what is contemplated is a gravity dam which by a large increase in mass and cross section would permit a substantial reduction in unit cement content and a great simplification of construction processes. As an average case (see Fig. 28), let it be assumed that the upstream face has a slope of 10 on 1, and the downstream face a slope of 1 on 1. Forms would be used on both upstream and downstream faces, but unless conditions of stream diversion necessitated otherwise there would be no joints from abutment to abutment, and the only restrictions to a contractor's free operations within the main boundary limits would be those imposed by the construction of inspection galleries, conduits, and similar elements.

For the main body of the dam a lean, damp, almost dry concrete would be used—0.50 to 0.60 bbl cement per cu yd supplemented with fines. This would be spread in thin layers and vibrated by bulldozers operating over the top surface of the dam as it was built up, sprinkling and water curing following as the work advanced over successive areas. By its low cement content such concrete, developing 2,000-lb to 2,500-lb strength in a few months, would automatically generate but one half to two thirds the heat of ordinary dam concrete and would have a correspondingly reduced tendency to volume change from that cause. Its modulus of elasticity likewise would be low, and therefore such concrete would the better accommodate itself to slight settlements and adjustments of foundations. If subjected to abrasion and erosion for short periods it would be adequately resistant, but not, of course, for months or years.

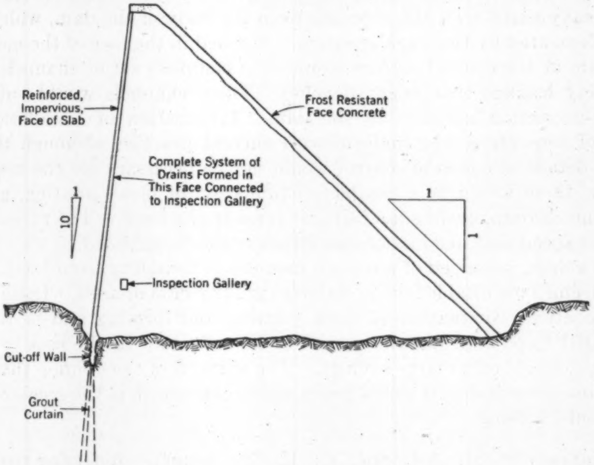
Reinforced, Impervious, Face of Slab
Frost Resistant Face Concrete
Complete System of Drains Formed in This Face Connected to Inspection Gallery
1
10
1
1
Inspection Gallery
Cut-off Wall
Grout Curtain

FIG. 28.—TYPICAL CROSS SECTION, CONCRETE FILL DAM

As stated, impermeability and durability would be provided at the exposed surfaces. For the downstream face a zone or layer of richer weather-resistant concrete integral with the main mass would be provided, as has been done in numerous instances in the past. At the upstream face would be located the water-stopping for the entire dam.

Two dams, both of which have their water-stopping (their impermeability) provided at the upstream face, have been described in the technical press.[113, 114] One of these is the repaired Ringedal Dam in Norway; the other is a steel-faced rock-fill dam near Colorado Springs, Colo. Both of these structures have impervious facings which transmit water pressures but at the same time cut off the penetration and passage of water to the main bodies of the dams. Quite similarly with the "concrete-fill" dam there would be provided a reinforced facing slab of suitably rich, impermeable concrete. This would lie directly against the main dam but would be separate from it, the contact surface being formed

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[113] *Engineering News-Record*, October 27, 1932, p. 498.

[114] *Civil Engineering*, January, 1939, p. 7.

as smooth as possible and asphaltic coated to minimize friction. For holding the facing slab to the main dam and affording lateral support against buckling under its own weight as a long column, anchors embedded both in the face slab and the dam would be provided, the connection being suitably detailed to permit slight vertical movements between the two. The face slab would connect with a heavy cutoff wall also separate from the body of the dam, which would be complemented by thorough grouting. Formed in the face of the main body of the dam at the contact surface would be a complete set of channels for collecting any leakage that might develop. These channels would outlet into drainage-inspection galleries in the dam. Preparation of foundations and control of concrete would conform with current practise, although the more rigorous details of concrete control would not be necessary for the main dam. However, there would be a complete elimination of block jointing, grouting, pour-plane cleanup, cooling, and similar present processes. The fullest opportunity for speed and freedom of operations would be afforded.

Such a dam, possessed of a certain measure of flexibility, could well be used at sites deemed unsuitable for the standard gravity dam or arch. In its lowered cement content, elimination of block jointing and forming and of the other enumerated steps and operations, and in its reduction in the cost of placing concrete, it would effect large savings. The writer is of the opinion that, where foundations are suitable, it would prove more economical, in the greater heights, than rolled-fill dams.

I. NELIDOV,[115] M. AM. SOC. C. E. (by letter).—Referring to "Basic Design Assumptions" so fully presented by Messrs. Houk and Keener, the writer wishes to discuss the following features of the design of high dams: The effect of Poisson's ratio on the stresses in straight and curved gravity and buttress dams, the validity of the shear-sliding ratio, and the major stress problems involved in raising a dam. The authors limited themselves primarily to discussion of dams of solid type. However, with buttress dams advancing into the high-dam class the writer saw fit to include them also in the discussion. The simplest theoretical deductions were used in order not to obscure the issue. Numerical values given in the discussion have only a relative meaning.

As is known, the true unit deformations, or deformations including lateral distortions due to Poisson's ratio, are expressed as follows:

$$e_x = \frac{1}{E}\sigma_x - \eta\,(\sigma_y + \sigma_z) \quad \text{.................. (17a)}$$

$$e_y = \frac{1}{E}\sigma_y - \eta\,(\sigma_x + \sigma_z) \quad \text{.................. (17b)}$$

and

$$e_z = \frac{1}{E}\sigma_z - \eta\,(\sigma_x + \sigma_y) \quad \text{.................. (17c)}$$

in which: σ = apparent stresses, with subscripts corresponding to three orthogonal axes; $\eta = 0.2$ = Poisson's ratio; and E = the modulus of elasticity. The apparent stresses, σ, are determined from boundary conditions of loading, equations of equilibrium, and the compatibility equation.

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[115] Senior Engr. of Hydr. Structure Design, Dept. of Water Works, State Div. of Water Resources, Sacramento, Calif.

It may be noted that true stresses based on unit deformations will be:[118]

$$\sigma_x' = e_x E \qquad (18a)$$

$$\sigma_y' = e_y E \qquad (18b)$$

and

$$\sigma_z' = e_z E \qquad (18c)$$

The following notations are used for the apparent stresses: At the upstream face, σ_x = the horizontal stress due to arch action; σ_y = the horizontal stress due to water pressure; and σ_z = the vertical stress due to water load and own weight of the dam. The true stresses, σ_x', σ_y', and σ_z', correspond to the foregoing apparent stresses. At the downstream face, σ_1 = the stress parallel with the face, acting in a vertical plane; σ_2 = the stress normal to the face; and σ_3 = the stress parallel with the face acting in a horizontal plane. The corresponding true stresses are σ_1', σ_2', and σ_3'.

The foregoing stresses are assumed to be equivalent to principal stresses so as to avoid introducing complication due to shears. It was assumed that there is no tension in the arch.

The effect on the stresses of an absolutely unyielding abutment was computed approximately by making $e_x = 0$ and determining σ_x or σ_3 from Eq. 17*a*. Curved gravity dams and buttress dams are considered.

The resulting stresses are given in Table 19, which shows that, at the upstream face, tension of 70 lb per sq in. exists in the direction of the arch, in case No. 2. This tension decreases to zero for the rigid abutment, in case No. 3. For a buttress in case No. 4, it becomes more dangerous since there is no lateral support.

TABLE 19.—COMPUTATION OF APPARENT STRESSES
($\sigma_y = 250$; $\sigma_z = 100$; $\sigma_1 = 600$; and $\sigma_2 = 0$, for each case)

Case No.	UPSTREAM FACE				DOWNSTREAM FACE				Remarks
	σ_x	σ_x'	σ_y'	σ_z'	σ_3	σ_1'	σ_2'	σ_3'	
									Curved Gravity Dam:
1	100	30	210	30	0	600	−120	−120	At crown
2	0	−70	230	50	100	580	−140	− 20	At abutment
3	70	0	216	36	120	576	−144	0	Right abutment
4	0	−70	230	50	0	600	−120	−120	Buttress dam

At the downstream face tensions exist also at the crown and, of the two tensions, $\sigma_2' = -120$ lb per sq in. is the most dangerous since it is acting away from the face that has no lateral support. At the abutment, tension in the direction of the arch is decreased due to the rigidity of the abutment, but it is increased in the direction normal to the face. In the case of a buttress (case No. 4) tensions of $\sigma_2' = \sigma_3' = -120$ lb per sq in. exist in the directions normal to the face and to the axis of the buttress. Both these stresses are dangerous since there is no lateral support to resist them.

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[118] "Mechanics of Materials," by the late Mansfield Merriman, M. Am. Soc. C. E., 11th Ed., John Wiley & Sons, New York, N. Y., p. 359.

If the tensile strength of concrete is 250 lb per sq in., the factor of safety is about: $\frac{250}{120} = 2.1$, which in itself might be regarded as sufficient provided one could rely on the uniformity of the strength of concrete. It may be noted that the factor of safety in compression for a 3,000 lb per sq in. concrete will be $\frac{3,000}{600} = 5$.

The writer believes that consideration should be given to the effect of Poisson's ratio when designing high concrete dams and especially those of the buttress type.

In commenting on the shear-sliding factor in computing stability of a gravity dam against horizontal forces, it may be noted that sliding cannot occur simultaneously with shear, but will begin only after the shearing failure has occurred. Consequently, it appears that only shearing resistance should be introduced in computing stability. At the same time, since sliding resistance is ordinarily smaller than shearing resistance, the minimum factor of safety should be based on sliding alone, and the maximum factor on shearing alone. Of course, this excludes the case in which the excavation profile is so rugged that no sliding can possibly occur. A comparison should be made in each case between sliding and shearing resistance. Assuming an average compressive stress of 100 lb per sq in., an area of the base equal to 200 sq ft, the friction coefficient equal to 1, and without going into the detail of computing shearing resistance based on actual stress distribution, the sliding resistance will be: $100 \times 144 \times 200 \times 1 = 2,880,000$ lb. The shearing resistance based on a shear of 200 lb per sq in. will be: $200 \times 144 \times 200 = 5,760,000$ lb. This indicates that sliding resistance controls in this particular example.

In recent years another problem has become connected with the design of high dams—the problem of either raising the existing dam, or providing for a future increase in height. The actual cases in which the heights of dams have been increased are not numerous. Among the more important are: Assuan Dam, in Egypt; Huntington Lake Dam, Sweetwater Dam, O'Shaughnessy Dam, and Alpine Dam, all in California. As might be anticipated, there are many design and construction questions involved in adding to the height of a dam. The method used at O'Shaughnessy and Alpine dams consisted principally of adding a slab of concrete to the downstream face of the existing dam, thus forming an inclined joint between the old and new concrete. In order that the dam might be monolithic this joint should be capable of transmitting the imposed stresses. In the O'Shaughnessy and Alpine dams the downstream faces of the existing dams were stepped (3.5 run and 5.0 rise). These steps were roughened by chipping.

In Fig. 29(*a*) the elementary cube is shown inscribed into the steps, with its proportions referred to the dimensions of the steps, and forces acting on its side. Thus p and p' denote normal stresses; q = shearing stresses; and f = principal stresses. Fig. 29(*b*) is Mohr's circle of stress; and a force diagram is shown in Fig. 29(*c*).

In case both vertical and horizontal steps are entirely ineffective in carrying shear, the forces $q_v \times 0.520$ and $q_h \times 0.364$ should be replaced by a force $A\,B$ = 98 lb per lin in. of length of the dam, acting at an angle 35° with the

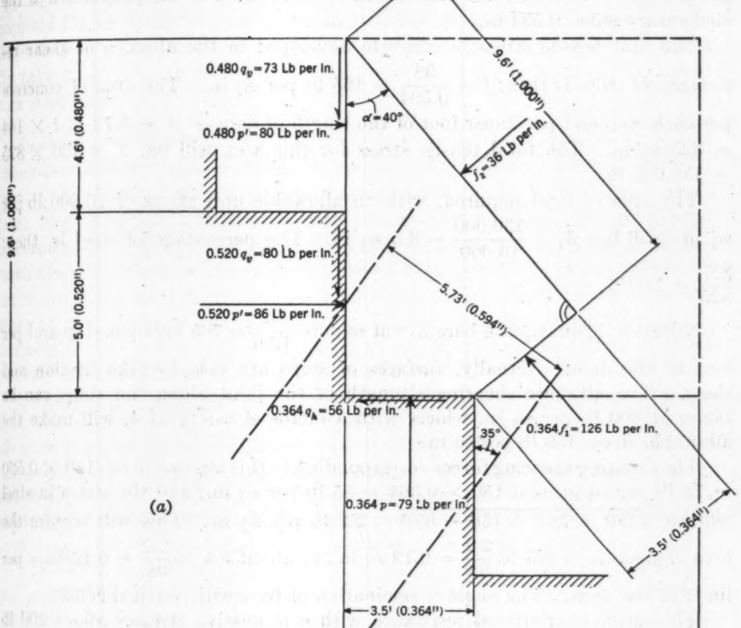
0.480 q_v = 73 Lb per In.
α = 40°
0.480 p′ = 80 Lb per In.
9.6′ (1.000″)
4.6′ (0.480″)
5.0′ (0.520″)
0.520 q_v = 80 Lb per In.
0.520 p′ = 86 Lb per In.
9.6′ (1.000″)
f_2 = 36 Lb per In.
5.73′ (0.594″)
0.364 q_h = 56 Lb per In.
35°
0.364 f_1 = 126 Lb per In.
0.364 p = 79 Lb per In.
3.5′ (0.364″)
3.5′ (0.364″)
9.6′ (1.000″)
(a)
q_v = 153
α = 40°
Direction of f_1
100
Values of q_v in Lb per Sq In.
f_1 = 346
p = 217
p′ = 166
f_2 = 36
0
Values of p, p′, f_1 and f_2 in Lb per Sq In.
100
Values of q_h in Lb per Sq In.
q_h = 153
(b)
f_2 = 36
p′ = 166
α = 40°
0.364 f_1 = 126
0.480 q_v = 73
35°
A
0.364 p = 79
98
0.520 q_v = 80
B
0.364 q_h = 56
(c)

FIG. 29

vertical. Since the steel will be placed at the middle of each rise of the step the area on which this force acts is 5.73 ft or, reduced to the proportion of the elementary cube, 0.594 in.

The unit tensile stress in concrete developed in the absence of shear resistance of steps is then: $f = \frac{98}{0.594} = 165$ lb per sq in. The area of concrete per each step and per linear foot of the length of dam is: $A = 5.73 \times 1 \times 144 = 825$ sq in. The total tensile stress for this area will be: $T = 166 \times 825 = 136{,}000$ lb.

The area of steel required, with an allowable unit stress of 16,000 lb per sq in., will be: $A_s = \frac{130{,}000}{16{,}000} = 8.5$ sq in. The percentage of steel is, then: $\frac{8.5}{825} = 1.03\%$.

Selecting 1¼-in. square bars it will require $\frac{8.5}{1.56} = 5.5$ bars per step and per foot of the dam. Actually, surfaces of steps are able to take friction and shear. The ultimate shearing strength of the joint along the steps can be taken at 600 lb per sq in. which, with a factor of safety of 4, will make the allowable stress 150 lb per sq in.

The resisting shearing forces corresponding to this stress will be: $150 \times 0.520 = 78$ lb per sq in. and $150 \times 0.364 = 55$ lb per sq in.; and the stress in steel will be $\sqrt{(80 - 78)^2 + (56 - 55)^2} = 2.2$ lb per sq in. This will require the area of steel $A_s = 8.5 \times \frac{2.2}{98} = 0.19$ sq in., or about $5.5 \times \frac{2.2}{98} = 0.12$ bars per lin ft of the dam. The angle of inclination of bars with vertical is 35°.

The ultimate frictional resistance with compressive stresses about 200 lb per sq in. may be assumed to be about 4, so that, with the factor of safety of 4, the allowable friction coefficient will be 1. This will produce the resisting frictional forces equal to $166 \times 0.52 \times 1 = 86$ lb per sq in. and $217 \times 0.364 \times 1 = 79$ lb per sq in. Since these forces are greater than the acting shearing forces, no steel is needed.

Another consideration enters the design of steel bars—it is the grouting pressure due to grout inserted in the joint after new concrete is poured, in order to make the joint tight. Assuming a grouting pressure of 50 lb per sq in., the total tensile force per step and per linear foot of the dam will be: $T = 50 \times 825 = 41{,}250$ lb. The vertical depth of the new concrete over the joint is $h = 113$ ft and its unit weight is 150 lb per cu ft. The component of its weight resisting the grouting pressure is: $P = 150 \times 113 \times 3.5 \times 1 \times 0.574 = 34{,}100$ lb. The remaining part of the grouting pressure, $41{,}250 - 34{,}100 = 7{,}150$ lb, must be taken by steel. If steel is placed at an angle of 35° with the vertical, the axial component of this force will be: $7{,}150 \times \cos 20° = 6{,}700$ lb, and the transverse component will be: $7{,}150 \times \sin 20° = 2{,}440$ lb.

The area of steel for tension is then: $A_s = \frac{6{,}700}{16{,}000} = 0.42$ sq in. per lin ft of the dam, or $\frac{0.42}{8.5} \times 5.5 = 0.27$ bar. The area of bars required for shear is then: $A_s = \frac{2,440}{12,000} = 0.20$ sq in. per lin ft of the dam. The total area of steel required for carrying the water load and resisting grouting pressure will be: $A_s = 0.19 + 0.42 = 0.61$ sq in. or $0.12 + 0.27 = 0.39$ bar per lin ft of dam. By taking 1½-in. square bars and spacing them on 2.5-ft centers, the desired area is obtained.

James B. Hays,[117] M. Am. Soc. C. E. (by letter).—To the statement about core drilling interpretations in the excellent paper by Messrs. Paul and Jacobs, the writer would like to add particular emphasis. The type of equipment best suited for maximum recovery should be used. Hydraulic-feed diamond drills will recover more of the softer forms of rock than the screw-feed machines. Continuous observation of the drilling, such as changes in color of the return drill water, the pressure or rate of feed on the drill itself, together with notes on the length of rods or depth of hole at each change, will be of considerable aid in preparing an accurate log of the hole when less than a complete recovery of core is brought up in the core barrel.

On the subject of drainage the writer would like to call attention to the fact that, in many instances in the past, drainage holes have been drilled into rock too close to a grouted cutoff curtain so that they penetrated rock that had been tightly grouted and did not function as drains. It is often the case that grout and drain holes are drilled from the same gallery. If this is done, the grout holes should be inclined in an upstream direction and the drain holes sloped downstream to clear the previously grouted zone or curtain.

The writer heartily agrees with Mr. Tyler in his paper on concrete control and would like to add some points on cracking from personal observations. Foundation cracks can be largely eliminated by placing concrete in thin layers on the rock and allowing time for heat loss. Reinforcement has also been used over sharp breaks in rock ledges. Horizontal joints also should be made at such points. Other cracks are caused by improper design of concrete sections. However, as Mr. Tyler states, 75% or more of the cracking is due to temperature effects.

A few years ago the writer observed that where forms were removed from mass concrete sections twenty-four hours after pouring, and water curing started immediately, the cracking was noticeably less than where forms were left in place for two to five days. Douglas McHenry prepared some computations along this line (unpublished) and came to the conclusion that the concrete would stand a far greater temperature drop without cracking at twenty-four hours than it would from two to five days later. Beyond this time conditions slowly improve so that after twelve or fifteen days the forms may be removed with about as much danger of causing the concrete to crack as if removed in twenty-four hours. It is imperative that the concrete be kept moist at all times. When forms are removed in twenty-four hours, water curing should be started immediately and should be maintained continuously for as long a period as possible. When wood forms are left in place for a long period, cracks often will be found if the concrete has dried out. The writer's experience along the foregoing lines covers two projects in which limestone coarse aggregate was used and the "pours" were in 10-ft lifts. Cracks have not been eliminated completely, but those few that have occurred are very fine and limited in extent.

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[117] Constr. Engr., Kentucky Dam, TVA, Gilbertsville, Ky.

When forms are left in place for a period of several days or longer, absorbent form-lining materials will keep the surface of the concrete moist and will prevent cracks caused by drying. Concrete surfaces poured against absorbent form lining resist the absorption of water to a high degree, and indications from observation are that the dense surface also loses less water from evaporation. Observation on concrete placed against an absorbent form lining denotes that almost no cracks occur, compared to a face placed against a good oiled-wood form. Absorbent form lining has been used at Kentucky Dam for several months and the results, so far, are very pleasing.

The writer would add three other items to the list of seven of the procedures tending toward crack elimination, as follows: (8) Early removal of forms and the immediate and continued application of curing water; (9) use of absorbent form-lining materials; and (10) selection of proper aggregate, if available.

Mr. Steele has given a thorough analysis of the problem of construction joints. The writer would like to add a few notes from his experience in grouting contraction joints, gained while working under Mr. Steele when considerable experimental work was being done along this line. The work at Boulder Dam was described[118] by the writer in 1937. Earlier methods used by the writer on the Calderwood Dam were described[119] in 1933.

Experience indicates that the use of horizontal grout stops should be avoided and the grouting layout arranged so that all joints can be grouted simultaneously from bottom to top. This is not always possible, but this fact need not prevent obtaining a satisfactory job. The elimination of the horizontal grout stops will permit the regrouting of lower lifts when the upper sections of the joints are opened during grouting. Horizontal grout stops placed above the outlet headers of the grouting system leave a space where thin, foamy grout accumulates and cannot be drained off. Horizontal stops will be necessary at the top of the dam, as well as under and over galleries. Since these stops are difficult to install, the writer prefers to follow a zigzag or sloping pattern. In this way there is only a short section of the seal covered by the concrete at any one time, and if the seal becomes bent down it will be indicated above the surface. There is also less chance of getting honeycomb concrete just under the seal. High, thin, arch dams must be grouted with extreme care to avoid excessive upstream deflections that might cause damage to the structure.

In comparing grouted joints with concrete filled slots, there is some advantage in favor of the grout, in that open cracks in the joint faces of the concrete will be filled.

James S. Lewis, Jr.,[120] Assoc. M. Am. Soc. C. E. (by letter).—The information contained in the comprehensive paper by Messrs. Paul and Jacobs might well be included in the curriculum of every engineering school. Too frequently, engineers confronted with foundation problems must secure such knowledge by painstakingly arriving at solutions that have been obtained previously by other engineers. Much time is thus lost and expensive effort needlessly duplicated. Messrs. Paul and Jacobs have made a valuable contribution to a technical subject that does not lend itself to expression in exact terms. Despite certain widely applicable fundamentals, every foundation presents questions which may not be answered until the true nature of the rock is revealed by excavation.

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[118] *Civil Engineering*, February, 1937, p. 126.

[119] *Loc. cit.*, November, 1933, p. 606.

[120] Constr. Supt., Watts Bar Dam and Steam Plant, TVA, Watts Bar Dam, Tenn.

Where the improvement of bearing strength is a serious consideration, it is frequently advisable to extend the excavation, under the gravity section of the structure, below any large horizontally disposed seams that would require filling in order to support the superimposed load. In addition to a tendency to become smaller, seams at greater depths have a tendency to pinch out and form contact areas with greater frequency, with the result that the inherent bearing strength of the foundation ordinarily becomes greater and depends less upon grouting as the depth increases.

Although natural irregularities frequently afford a condition which makes additional bond unnecessary, this is not always true, and, as Messrs. Paul and Jacobs have indicated, shale may pose a particularly difficult problem in this respect. The navigation lock of the Watts Bar Project rests upon a soft, blue, fissile shale, which contains thin layers of interbedded sandstone. In most areas under the lock, the dip was found to be quite gentle, and in order to obtain bond it was necessary to trim the surface of the shale to form a series of serrations running parallel to the strike of the rock. The profile of a line taken normal to the strike would have an irregular saw-toothed appearance, with the height of the teeth varying from approximately 6 in. to 3 ft. This work was done by hand, and the use of power tools was held to a minimum to avoid disturbance of the remaining rock.

Drains formed by half-round pipe or inverted wooden troughs on the surface of the foundation rock may be plugged with mortar when covered with concrete, unless exceptional care is exercised, as vibration of the concrete reduces the mortar to a highly plastic state, enabling it to flow through extremely small openings. Relatively shallow drain holes, drilled through the concrete later to supplement the built-in drainage system, offer good insurance against excessive uplift and may be found especially valuable under aprons or other thin sections.

Messrs. Paul and Jacobs have emphasized the importance of skilled interpretation of the information obtained from core borings, and the writer firmly believes that the best talent obtainable for this purpose is the cheapest. All too frequently, poorly paid, unqualified men, lacking in interest, training, and experience, are used as inspectors of core drilling operations. The result is that they serve as nothing more than recorders, and the records are of little value. One trained engineering geologist is of greater value than a dozen such men, and the wages of a qualified man will be repaid many times over later when construction is under way.

For exploratory work, the diamond core drill is far superior to the shot drill. The core loss is less, and that core which is recovered retains its original appearance to a greater degree. As the diamond drill is superior to the shot drill, so is the double-tube diamond core barrel superior to the single-tube barrel.

Blasting for rough excavation should be carefully controlled in order to reduce the expense attendant upon the removal of damaged rock outside the required line of excavation; Messrs. Paul and Jacobs recommend that drilling be limited to a specified proportion of the total depth in order to avoid damage to the rock below. The reference is probably to operations which involve the drilling of large areas. It is frequently practical to develop a vertical face, or series of faces, in the area of excavation, so that the force of the blasts is relieved outwardly with no damage to the rock below. When this can be done, the blast holes may be drilled the full depth of the proposed excavation with no adverse results.

The shale which composes the foundation of the Watts Bar Project navigation lock was excavated with power shovels, without resort to blasting, to within limits set 1 ft above contemplated grade and 1 ft inside of final lines. The unit cost of this rough excavation was little more than that of digging compact earth, but, by the time the hand work that was required to remove the remaining surplus material was completed, the unit cost had increased several fold to a figure which was representative of the cost of removing hard rock.

It is timely that Messrs. Paul and Jacobs should point out that line drilling is an effective method of accurately confining excavation to prescribed limits, when this is necessary, and that it is also expensive. The writer would like to emphasize the last portion of this statement and to call attention to the fact that the cost of line drilling easily may exceed the presumed value of any concrete saved by preventing overbreakage of the rock. When design considerations dictate the need for a clean, vertical face, line drilling is unquestionably justified, but considerable discretion should be exercised in specifying it for the purpose of effecting economies. Much line drilling has been done to no effective purpose.

CHARLES H. PAUL,[121] AND JOSEPH JACOBS,[122] MEMBERS, AM. SOC. C. E. (by letter).—The writers are glad to note that all of the discussers are practically in agreement as to the correctness of the principal suggestions and assertions of the paper. The writers are in full accord with Mr. Riegel in his statement that "The most competent and continuous supervision possible should be a feature of dam site exploration." Vigilance and competent judgment are essential throughout the entire schedule of foundation preparation, and the exploratory period is, by no means, exempt from these requirements. To Mr. Riegel's suggestion that deep seating of the dam structure is aimed at utilizing shearing strength to develop resistance against lateral movement, the writers would repeat what is already stated in the paper—namely, that it is also aimed at providing, at the toe of the dam, direct compressive resistance to lateral movement.

Mr. Jones' discussion is mainly a plea in behalf of lower grouting pressures, indorsing and emphasizing the writers' advocacy of the same consideration, and he supports his emphasis splendidly with sound argument and illustration. The plea for lower grouting pressures is timely and highly desirable. There is some contention that the higher pressures, with their resultant slight rock displacements, are not objectionable, in that they insure a more definite injection and a more solid and continuous placement of the grout. Something may be said for this contention but the writers, and apparently Mr. Jones also, favor the more conservative policy of limiting the pressure sufficiently to insure against possible rock displacement. It is believed that the other procedure involves some elements of uncertainty and hazard in that it may unduly widen and extend existent fracture planes and possibly open up new fracture planes. Mr. Jones' description of the uplift gage used at Norris Dam and his discussion of the need of such gages in general are of great interest.

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[121] Cons. Engr., Dayton, Ohio.

[122] Cons. Engr., Seattle, Wash.

Mr. Binnie's experience at Silent Valley Reservoir in finding preliminary borings entirely deceptive as to the actual position of bedrock is, by no means, an unusual experience. The history of dam site explorations is replete with such occurrences. It emphasizes the necessity of realizing in advance that preliminary borings may be deceptive and, for important structures, the necessity of making such borings quite thorough and complete. Fully as dangerous as the boulder deception is that of a relatively thin stratum of rock which, for lack of deep drilling, is assumed to be the true ledge bedrock. In morainal foundations it is important, when bedrock is apparently encountered, to continue the drilling to a depth somewhat greater than the possible thickness of boulders in that particular morainal deposit, relying upon the best judgment of competent geologists as to what that further depth should be.

Mr. Lewis' reference to the preparation of the foundation for Watt's Bar navigation lock is a good illustration of modern practice in preparation of shale foundations.

His advocacy of the diamond core drill is somewhat broader than the writers could express it. Where the character of the rock is such that large cores (3 in. to 5 in.) are required for most complete recovery, the shot drill has its place and often will yield a higher percentage of core at lower cost. Skilful use of the most suitable equipment is the main consideration. It should be borne in mind constantly that the only purpose of exploratory drilling, and the only justification for the cost involved, is to determine, as fully as is practicable, the actual nature of subsurface conditions. The first step in that determination is to secure the highest possible percentage of core recovery. Then follows the required correct interpretation of drilling experience and results, the importance of which can scarcely be overemphasized.

The writers agree with Mr. Lewis as to a permissible closer drilling for excavation of vertical faces where no damage to the rock below can be involved. They also are in accord with his observations on line drilling. With respect to all such matters, good judgment must be used both in the preparation of specifications and in the actual construction operations. Where some overbreakage is not in itself objectionable structurally, and the cost of the extra concrete involved is less than the extra cost of line drilling, then, of course, line drilling should not be used.

It is noted that Mr. Hays concurs in the writers' statement as to the importance of correct core drilling interpretations. His comments on the selection of suitable equipment, and on some of the details of continuous observation during drilling operations, are constructive and entirely correct. Regarding the relation of contiguous lines of grout holes and drainage holes, the relative inclinations of the holes as outlined by Mr. Hays are now the usual practice. As to the possibility of too close proximity, it may be observed that, while they should be far enough apart to avoid interference, their basic purpose requires that they both be located relatively near the upstream face of dam—the grout curtain to prevent, as far as practicable, initial seepage under the dam, and the drainage holes to catch that seepage as early and as far upstream as possible so as to reduce upward pressure to a minimum.

IRVING B. CROSBY,[123] AFFILIATE AM. SOC. C. E. (by letter).—Mr. Nickell and Mr. Moneymaker have effectively supplemented this writer's outline (which could not cover all details on account of limitations of space), and they have also discussed the methods of the Bureau of Reclamation and the Tennessee Valley Authority. The examples described by the writer were given as illustrations of the types of problems most commonly encountered with the different rocks—not as explicit instructions for the handling of specific conditions. Every dam site is a new problem requiring individual study; no two are exactly alike. The use of analogy may be helpful where it is difficult to obtain all the facts, but it may be dangerous due to the infinite variety of geological conditions. The omission of dams on pervious foundations of sand or other soils was intentional, since the adequate discussion of this case would have greatly extended the paper.

The writer is heartily in accord with the practice of the TVA of assigning work in geophysics, soil mechanics, and allied sciences to specialists in those fields. The geologist should recognize those problems, however, call attention to the need of allied specialists, and then outline the subsurface conditions and interpret the results. This is necessary because the work of these specialists can be no more accurate than the interpretations of geologic conditions to which it is applied.

Although a geological investigation has not always insured against difficulties with dam sites, increasing experience and added facilities of investigation have reached the point where the danger of disaster should be negligible provided the geologists are sufficiently experienced and receive adequate cooperation from the engineers. The value of geological investigations of dam sites is accepted generally; but the idea (still held by some) that geological defects are unpredictable is erroneous and pernicious. Geological conditions can be determined in detail if sufficient investigation is made. It may be difficult or uneconomic to do so, but even in the most difficult cases where it may not be feasible to determine all details, the geologist can outline the limiting possibilities and explain the conditions that must be met.

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[123] Cons. Eng. Geologist, Boston, Mass.

I. L. TYLER,[124] M. AM. SOC. C. E. (by letter).—Those who have commented on the paper deserve credit for their interest and for their aid in attempting to bring to the attention of engineers generally the importance of proper control of concrete and concreting procedures, particularly on projects in which the structures are expected to be reasonably permanent. It is readily agreed that there is still much to be learned about concrete in all its phases, but it is certain that better use of what is known would go far toward bringing average concrete construction to a level of quality well above that which now exists. If the paper and the comments have aided in this respect, the writer is indeed gratified.

The problem of maintaining uniformity of concrete is one of the most difficult encountered once construction is under way. Mr. Tuthill's comment should furnish "food for thought" along the very troublesome line of aggregate-grading control, which is so easily handled in the office, but which is entirely another problem on the job.

Mr. Kitts apparently disapproves the generally accepted trend of thought on mass concrete, classifying it as a product of a misguided "school" of thought, although it is doubtful that the viewpoints are really so far apart as might at first appear. Mr. Kitts' attraction to absolute volume measurements appears to illustrate a general impression that the sum of the absolute volumes of cement, water, and aggregates always equals the volume of mixed concrete. In few, if any, cases is this exactly true; in some cases it is not even a good approximation, even for plastic mixes. Effects of bleeding, evaporation, entrained air, and other variables combine to nullify, not the principles to which Mr. Kitts refers, but an appreciable part of their usefulness in calculating concrete mixes. Absolute volumes, however, are almost always used, at least in preliminary studies of concrete mixes. Even in setting the weight-batcher scales, the concrete engineer is thinking about an absolute volume when he adjusts the weighing apparatus to deliver, say, 1,675 lb of material.

Consistency of concrete must of necessity be governed by placing conditions and in this respect, if in no other, the "old army specification," as quoted by Mr. Kitts, fails entirely. (Inspection of any current construction project under the direction of the Army Engineers suggests that these engineers are not familiar with the "specification" attributed to them by Mr. Kitts.) The proposal of Mr. Binnie to use very thin layers of extremely dry concrete compacted by surface vibration is interesting in this connection. One of the oldest (and best preserved) dams in the United States was constructed in a similar manner, steel tamps operated by hand taking the place of the pneumatic hammers. Whether this procedure could be used with the present demand in the United States for speed of construction is a little doubtful, but the performance of dams built in this manner compared with those of the chuting era, which Mr. Kitts seems to prefer, is an indication that the present tendency in mass concrete is at least in the right direction.

Comments by Mr. Blanks need no amplification except possibly to observe that if construction joints were not required one serious problem in dam construction would be just about eliminated.

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[124] Research Engr., Portland Cement Assn., Chicago, Ill.

Control of cracking has been commented on by Messrs. Hadley, Hays, and others, in connection with this paper of the Symposium and by several in discussions of the papers on dam design. It has been pointed out that certain types of cracks in some locations of a structure may have little or no effect on stability, and it has been suggested that possibly too much attention is being given to the control of cracking. A review of the designer's position regarding cracking reveals considerable concern, however, and the very fact that the effect of most cracks on the stability of a structure cannot be determined appears to be sufficient reason for crack elimination if design and construction procedures will permit this at a reasonable cost. Present indications are that much can be done along this line without greatly increasing construction costs.

Byram W. Steele,[125] Assoc. M. Am. Soc. C. E. (by letter).—The various discussions relative to construction joint problems have given evidence that there is a growing interest in the elimination of the objectionable results that too often accompany joints of any kind in concrete. The solution of most of these problems must be demonstrated over and over again in the realm of actual structure performance before they will be accepted as the basis of design assumptions. The number of variables is too great to permit of an easy mathematical treatment. What is urgently needed is more field study and analysis of actual structure performance instead of the acceptance of design criteria based on assumed theoretical conditions that are not founded on facts.

It is desired to call attention to the point which Mr. Hays made in his discussion relative to the effect of removal of forms in twenty-four hours on surface cracking. Mr. Hays has made a valuable observation and one that should be given careful consideration because of the possibilities for crack control, providing construction difficulties can be eliminated. It is often difficult to remove forms in twenty-four hours, or early enough to control surface and near-surface temperatures properly; and it is also not economical to leave forms in place long enough to be of any particular benefit. However, the detrimental effect of high-surface and near-surface temperatures must not be ignored or minimized if surface-crack inception is to be prevented. If forms were so designed that they could be loosened at twenty-four hours to begin the spraying of concrete with water, it undoubtedly would be worth the cost in crack elimination; but it is so easy to mar the surface or break chunks out of chamfers and edges in such green concrete that, to date, early removal of forms for mass concrete has not been permitted.

In closing this discussion on construction joints the writer urges that the study of joints and cracks be given a major rôle in the future programs of field and laboratory research to the end that joints may be designed in so far as practicable to displace volume-change cracks that cannot be eliminated, and that cracks—however insignificant—be discouraged from starting by every means that is within the realm of economic practicability.

It is not beyond the realm of probability that within a few years engineers will be specifying that cement and aggregate shall be so selected that the resulting concrete will withstand the temperature changes produced by an extremely rapid rate of freezing and thawing during several hundred cycles without showing any evidence of disruption. Such a requirement would demand that the volume-change characteristics, whatever they may be called, of the various ingredients in the concrete shall be practically identical.

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[125] Head Engr., Office, Chf. of Engrs., War Dept., Washington, D. C.

The writer cannot agree with Mr. Hadley's philosophy of the harmlessness of a great deal of the present-day cracking. Surface cracks are the entering wedges of wholesale disintegration, and many of them—in fact, too many—cannot be considered otherwise than the indication of lack of care and attention to details. If crackless, or at least near-crackless, dams did not now exist this might be classed as rather a bold statement, but since there are a few specimens approaching that state of perfection it appears that the technique of producing crack-free structures is within the reach of the art.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 2122

TRANSATLANTIC SEAPLANE BASE, BALTIMORE, MARYLAND

By W. Watters Pagon,[1] M. Am. Soc. C. E.

With Discussion by Messrs. Karl Terzaghi, LeRoy L. Odell, and W. Watters Pagon.

Synopsis

Two major topics are described in this paper: (*a*) The difficult foundation conditions; and (*b*) the unusual hangar design dictated by these conditions, caused by the character of the mud fill which was originally placed in the Baltimore Municipal Airport, in Baltimore, Md. The paper deals only with the seaplane base, the first portion of which comprises a hangar 270 ft by 190 ft with 35 ft clear height, built entirely of steel, and a brick office building in the form of a half octagon, 66 ft by 125 ft, both of which are leased to the Pan American Airways System.

Brief History

First Stage.—In 1928 the voters of Baltimore approved the expenditure of $1,500,000 for the construction of an airport, which must be a combination landplane and seaplane base. It represented a recrudescence of the spirit which produced the famous Baltimore Clippers of one hundred years ago. In 1930 an additional $2,500,000 was voted. The site selected for the airport, although ideal from the operating standpoint and unexcelled elsewhere (see Fig. 1), was exceedingly poor from the engineering and financial standpoint. Of the 360 acres in the site, only 70 were fast land, and the remainder was open water having depths ranging from 6 ft to 18 ft. There were also 71.5 acres just outside of the field for use as automobile parking lots. The bed of the Patapsco River here consists of two, or more, marsh deposits, which are indicated by fossil leaves to be of Pleistocene (Talbot) age, and which extend nearly to 60 ft below mean low tide.

The site is on the north shore of the Patapsco River, about 5 miles in a direct line, or 6 miles by automobile, and 15 min from the Civic Center. It

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Note.—Published in October, 1940, *Proceedings*.

[1] Cons. Engr., Baltimore, Md.

is well served by highways, and is bounded on one side by a wide highway. A steam railroad freight line and a street-car line, which also serve the steel mills at Sparrows Point, Md., are nearby. Beyond the car line is the village of Dundalk, Md. The other three sides are bounded by open water, except for a portion of the south side, where there is abutting industrial property. A considerable portion of the area of the field is beyond the pierhead line, by special federal permission.

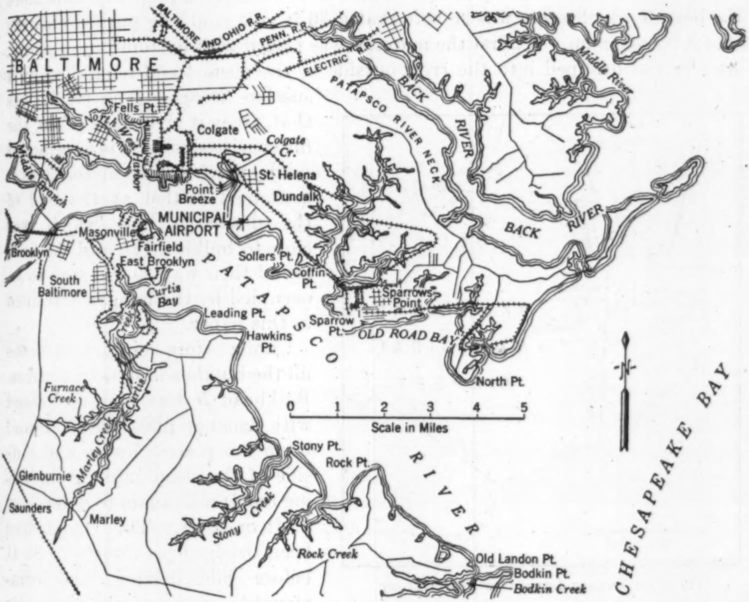

BALTIMORE
BALTIMORE AND OHIO R.R.
PENN. R.R.
ELECTRIC R.R.
PATAPSCO RIVER NECK
BACK RIVER
Middle River
Fells Pt.
North West Harbor
Middle Branch
Colgate
Colgate Cr.
St. Helena
Point Breeze
Dundalk
MUNICIPAL AIRPORT
Masonville
Brooklyn
Fairfield
East Brooklyn
South Baltimore
Curtis Bay
Sollers Pt.
Coffin Pt.
Sparrows Point
BACK RIVER
Leading Pt.
Hawkins Pt.
Sparrow Pt.
OLD ROAD BAY
North Pt.
PATAPSCO
Creek
Furnace Creek
Curtis
Marley Creek
0 1 2 3 4 5
Scale in Miles
N
Glenburnie
Saunders
Marley
Stony Pt.
Rock Pt.
Stony Creek
Rock Creek
RIVER
Old Landon Pt.
Bodkin Pt.
Bodkin Creek
CHESAPEAKE BAY

Fig. 1.—Location of Airport

As shown in Fig. 1, the Patapsco River is an estuary which opens into Chesapeake Bay and provides a minimum seaplane water runway 2 miles long directly in front of the airport, in a northeast-southwest direction, with almost unlimited length in other directions.

Along two sides of the airport permanent bulkheads, called *A*, *B*, and *C*, were constructed, the third side being bounded merely by a "mud fence," called *D* (see Fig. 2). Before construction of bulkhead *B* the river bed was dredged to 35 ft. The type chosen is an apron 38 ft wide, supported on plumb piles and two rows of batter piles, and surmounted by a concrete face wall with its top at 7.0 ft above mean low tide (see Fig. 3). A riprap toe was planned, but not placed.

Data for the wind rose in Fig. 2 were supplied by the U. S. Weather Bureau. The prevailing winds are southwesterly, the highest average velocity is north-westerly, and there is no perceptible wind 1% of the time. The normal tide range is 13 in.: high tides rise at times (due to wind) to +5.0 ft; in the tropical hurricane of October, 1933, the maximum height reached was +8.0 ft. Hence the hangar floor is at El. +8.0 and the office building floor at +9.0.

Material being excavated from a nearby ship anchorage was first planned for use in the airport, at a considerable saving, but the material that was actually received proved to be of very poor character and yielded disappointing results, as described subsequently. Other mud came from points in Baltimore Harbor, of which some was so soft that a 30-ft pole could be pushed entirely into it by one man. At first the material was placed with bottom-dump scows; later it was dropped into the river outside of the airport and rehandled by a suction dredge. The result was that, when it arrived in place, the fill consisted mostly of semiliquid mud. Such coarse material as it contained settled at the end of the discharge line, which was close to bulkhead *D*, and the fines found their way to the area to be occupied by the hangar described in this paper.

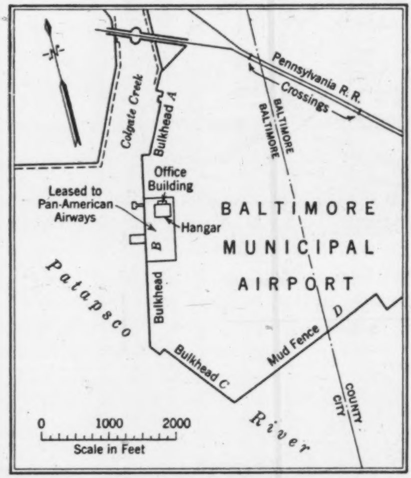
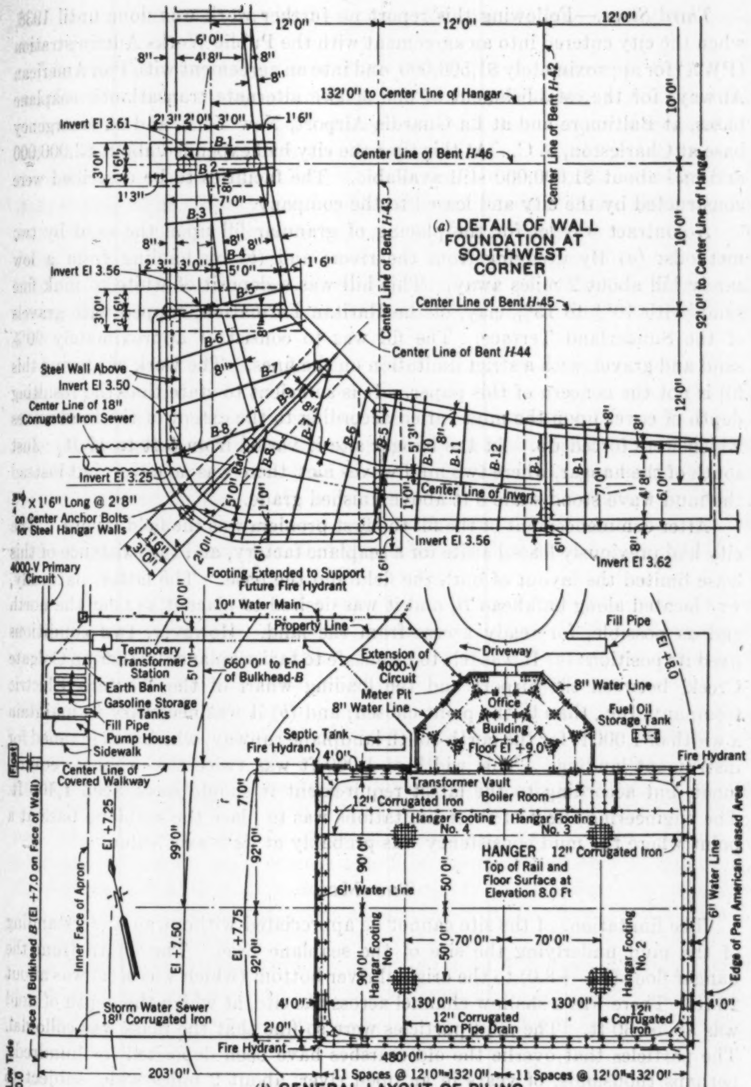


FIG. 2.—GENERAL LAYOUT OF THE AIRPORT

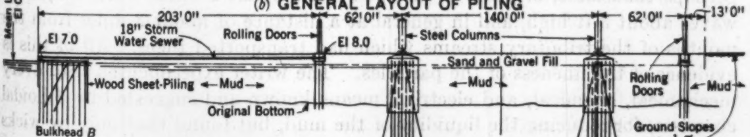
Long before completion of the fill the bulkheads showed distress. Bulkhead *B* was first reinforced with "anchor piles"; later a sand berm was placed, inside and outside of the sheet piling, thus filling one trench again to a depth of 15 ft or 20 ft. This trench had been dredged previously to 35 ft before pile driving was commenced. Some of the mud also flowed into this trench before the sheeting was completed. Bulkhead *C* was only a line of steel sheet piles, anchored back to deadmen, but, due to corrosion and to the loss of several anchorages, it has been buried by constructing a beach in front, riprapped with old paving blocks.

Second Stage.—In 1931 the city administration sought engineering advice as to what to do with the project. By that time the site had been almost completely filled in. The project constituted a mud fill so liquid that it had overflowed bulkhead *B* on at least two occasions, and had then become quiescent at an angle of repose of approximately 6 ft in 2,600 ft, or about 0° 8′. The answer given by the consulting engineers was that there was nothing that could be done except to let the fill lie in the sun and dry out for a period of twelve to fifteen months, and then to place about 3 ft of porous, sand-clay material on top of the mud by pumping a granular fill from the river bed.


(a) DETAIL OF WALL FOUNDATION AT SOUTHWEST CORNER

12'0"
12'0"
12'0"
6'0"
8"
4'8"
8"
8"
132'0" to Center Line of Hangar
10'0"
Invert El 3.61
2'3" 2'0" 3'0"
1'6"
B-1
Center Line of Bent H-46
3'0"
1'6"
B-2
8"
1'3"
7'0"
B-3
Center Line of Bent H-43
Center Line of Bent H-42
Center Line of Bent H-43
10'0"
92'0" to Center Line of Hangar
8"
8"
8"
B-4
2'3" 3'0"
1'6"
Invert El 3.56
5'0"
B-5
3'0"
1'6"
Center Line of Bent H-45
B-6
8"
Center Line of Bent H-44
Steel Wall Above
Invert El 3.50
B-7
Center Line of 18" Corrugated Iron Sewer
B-9
B-7
B-8
B-6
B-5
5'3"
B-10
B-11
B-12
B-1
B-1
8"
8"
12'0"
B-8
Invert El 3.25
5'0" Rad
7'0"
4'8"
6'0"
Center Line of Invert
3/4 x 1'6" Long @ 2'8" on Center Anchor Bolts for Steel Hangar Walls
1'0"
1'0"
2'0"
1'6"
3'0"
1'6"
Invert El 3.56
6"
8"
Invert El 3.62
4000-V Primary Circuit
Footing Extended to Support Future Fire Hydrant
10'0"
10" Water Main
Property Line
Fill Pipe
Temporary Transformer Station
5'0"
660'0" to End of Bulkhead-B
Extension of 4000-V Circuit
Driveway
Earth Bank
Gasoline Storage Tanks
Fill Pipe
Pump House
Sidewalk
Meter Pit
8" Water Line
Office Building
Floor El +9.0
8" Water Line
Fuel Oil Storage Tank
13+81'0"
Septic Tank
Fire Hydrant
4'0"
Fire Hydrant
Float
Center Line of Covered Walkway
Transformer Vault
Boiler Room
7'0"
12" Corrugated Iron
Hangar Footing No. 4
Hangar Footing No. 3
HANGAR
12" Corrugated Iron
Top of Rail and Floor Surface at Elevation 8.0 Ft
El +7.25
99'0"
92'0"
90'0"
50'0"
6" Water Line
6" Water Line
Edge of Pan American Leased Area
Face of Bulkhead B (El +7.0 on Face of Wall)
Inner Face of Apron
El +7.75
El +7.50
92'0"
Hangar Footing No. 1
90'0"
70'0"
70'0"
50'0"
Hangar Footing No. 2
4'0"
130'0"
130'0"
4'0"
Storm Water Sewer 18" Corrugated Iron
12" Corrugated Iron Pipe Drain
12" Corrugated Iron
2'0"
Fire Hydrant
480'0"
203'0"
11 Spaces @ 12'0"=132'0"
11 Spaces @ 12'0"=132'0"

(b) GENERAL LAYOUT OF PILING


Mean Low Tide El 0.0
203'0"
62'0"
140'0"
62'0"
13'0"
El 7.0
18" Storm Water Sewer
Rolling Doors
El 8.0
Steel Columns
Sand and Gravel Fill
Wood Sheet-Piling
Mud
Mud
Rolling Doors
Mud
Original Bottom
Bulkhead B
Ground Slopes Up 1/2 %

(c) SECTION NEAR SOUTH WALL OF HANGAR

FIG. 3.—FOUNDATION PLAN OF SOUTH WALL OF HANGAR

Third Stage.—Following this report no further work was done until 1936, when the city entered into an agreement with the Public Works Administration (PWA) for approximately $1,500,000, and into an agreement with Pan American Airways for the establishment of one of two alternate transatlantic seaplane bases, at Baltimore and at La Guardia Airport, New York, and an emergency base at Charleston, S. C. At this time the city had expended about $3,000,000 and had about $1,000,000 still available. The facilities to be described were constructed by the city and leased to the company.

A contract was let for the placing of granular fill upon the mud by two methods: (*a*) By dredging from the river; and (*b*) by hauling from a low sandy hill about 2 miles away. This hill was a deposit of white to pink fine sand, with 10% to 15% clay, of the Raritan Formation, capped with gravels of the Sunderland Terrace. The fill was to consist of approximately 90% sand and gravel, with a strict limitation on the fines. The work of placing this fill is not the concern of this paper. It is sufficient to state that the resulting depth of cover upon the mud varied according to the extent of the mud waves which were forced up. At the hangar site it varied from 5 ft to 17 ft. Just south of the hangar, where two mud waves met, there was no cover, but instead the mud wave stood some 5 ft above finished grade.

After commencement of the fill the first problem was the field layout. The city had previously leased a site for a seaplane factory, and the existence of this lease limited the layout of both the field and the base. The latter, naturally, was located along bulkhead *B*, and it was desired to place it as near the north end as possible, for ready access from the land. However, two conditions fixed its position: (*a*) It was felt to be unsafe to taxi seaplanes farther up Colgate Creek, between the airport and the loading wharf of the Western Electric Company, Inc., than to the point chosen; and (*b*) it was necessary to maintain a width of 1,000 ft for the north-south landplane runway, which was planned for instrument landings. This width of 1,000 ft was twice the original requirement, but according to the latest requirement it should have been 1,400 ft. The engineering effect of these limitations was to place the seaplane base at a point where the mud consistency was probably at its lowest value.

THE MUD

The limitations of the site cannot be appreciated without an understanding of the mud underlying the site of the seaplane base. The depth from the hangar floor (El. +8.0) to the original river bottom (which was soft) was about 26 ft. There was a shallow channel across the site, at which the depth of mud was about 30 ft. The mud particles were so fine that the mass was colloidal. The particles that overlie the old marshes have been deposited for hundreds, perhaps thousands, of years in an open river, about 2 miles wide, subject to waves about 3 ft high, and in general at a distance of about 3 miles from the mouths of the tributary streams which had transported them. All of this is evidence of the fineness of the particles. The writer experimented with every mechanical, chemical, and electrical means known and suggested by colloidal chemists, for reducing the liquidity of the mud, but found that only by wicks could the colloidal structure be destroyed. This idea formed part of the basic design concept.

The field slope was 1.0% (later 0.8%) upward from bulkhead *B*, for surface runoff. The runway is approximately 600 ft from the hangar, and is intended for instrument landings. The stability of the fill, therefore, was reduced by this surface slope. There was also the possibility of lateral displacement of the surface, due to jarring of landing airplanes, and the doubt, on the part of many of the public, as to the safety of bulkhead *B*, in which doubt, however, the writer did not concur. The first two reasons were sufficient for the foundation design that will be described. The cost of each main column foundation was nearly $20,000, and this would vary only a few per cent with the amount of column load; hence, there was introduced, also, a potent financial reason for the design.

When the work of filling, under "Third Stage," was begun, the mud fill had shrunk vertically from 3 to 5 ft, the exact amount being impossible to determine. Shortly after it had been pumped in, walking on its surface was impossible, because it could be stirred readily with a 6-ft stick. After eighteen months of drying in the sun there was a "crust" of indeterminate depth. If the depth to the bottom of the mud cracks is used, it was about 8 to 10 in.; but below the cracks was about 6 to 10 in. of putty-like mud. The topmost portion had dried to such an extent that it had reached its shrinkage limit, and then, on further loss of water, had turned from dark gray to a dirty white color. On the average the cracks had a top width of about 3 in., and occurred (in crude hexagonal pattern) on the average about every 10 in., indicating a linear contraction of perhaps 30%. The width of crack varied almost linearly down to the point where it closed, at which point the material was somewhere in the vicinity of the plastic limit.

Walking over this 18-month old cracked crust, with the underlying mud yielding under the weight at each step, created the sensation of walking on a tight bedspring. By jumping up and down at the proper frequency the entire area of crust, for a distance of 50 ft or more, could be set in oscillation. Under such forces the mud acted like an elastic jelly, and there was no evidence of permanent strain or set. When a chunk of the drier portion was lifted out, the piece would have a total depth of nearly 12 in., and the bottom break occurred where the consistency was about that of putty when it is dry enough not to adhere to the hands.

In 1931 the writer had had tests made by the U. S. Bureau of Public Roads on two mud samples and one sand sample from the berm under bulkhead *B*. The results were as given in Table 1, the particle size being graded as follows:

Grade	Size in millimeters (see Table 1)
Coarse sand	2 to 0.25
Fine sand	0.25 to 0.05
Silt	0.05 to 0.005
Clay	less than 0.005
Colloids	less than 0.001

Some unit weights of the mud were:

Moisture	Weight, in lb per cu ft
Dry	88
32% moisture	115
Plastic limit	110
Liquid limit	93

TABLE 1.—Soil Tests by the U. S. Bureau of Public Roads

Sample No.	Particle Size, in Millimeters (Percentage of Moisture)							Type of material	Liquid limit	Plasticity index	Shrinkage		Moisture Equivalent		Specific gravity	Limit shrinkage	Volume change
	>2	2 to 0.25	0.25 to 0.05	0.05 to 0.005	<0.005	Total <2	<0.001				Limit	Ratio	Centrifugal	Field			
S-6265	0	1	7	23	69	100	32	mud	84	44	32	1.4	95[a]	60	2.64	11	39
S-6266	0	2	1	26	71	100	31	mud	85	43	31	1.4	91[a]	61	2.65	11	42
S-6264	46	68	12	11	9	100	7	sand	..	..	..	..	..	..	..	..	..

[a] Waterlogged, indicating a "muck" soil.

Below the plastic limit the material develops some resistance to pressure, the critical point which marks the change from a fluid to this state being generally at about the plastic limit. Since plastic limit equals liquid limit minus plasticity index, the plastic limit was 40% and 42%, respectively.

An interesting comparison is afforded by tests[2] at the National Bureau of Standards, in which it was found that a "Maryland Clay," when extruded from a $2\frac{9}{32}$-in. round cylinder, through a $\frac{1}{2}$-in., sharp-edged die, showed zero pressure at a water content of about 44% of the dry weight, and for less moisture showed a hyperbolic curve of pressure versus moisture. At 18% moisture the pressure reached 400 lb per sq in.

One sample, semiliquid, had been taken from within the area of the seaplane base; the other, from within the crust, was taken from about 1,000 ft south. The results of the compression versus load, and compression versus time, tests are given in Figs. 4 and 5. A similar sample placed in an open-mouth jar, with a protruding and overhanging wick, dried at office temperature in a few weeks to about one third its original cubic volume, whereas a companion sample, without wick, was little changed at the end of six months. Water dripped from the end of the wick. When the mud is at its driest condition its consistency is nearly that of slate, and it will take a high polish.

Conditions at the Seaplane Base Site

When work on the seaplane hangar was begun, under the third stage of the work, the following facts and conditions obtained:

(*a*) The granular fill over the building area had a depth of 5 to 17 ft;

(*b*) There were unfounded fears on the part of many citizens that bulkhead *B* could not be regarded as safe;

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[2] "Relation Between Moisture Content and Flow-Point Pressure of Plastic Clay," by Ray T. Stull and Paul V. Johnson, *Research Paper RP 1186*, *Journal of Research*, National Bureau of Standards, Vol. 22, January–June, 1939, p. 329.

(*c*) The process of placing the granular fill at this site had pushed a mud wave across the site, such that a 1-in. steel pipe, left over the week end from a wash boring operation, was found to be 15 ft (at the top) from its original position (see Fig. 6);

(*d*) The writer was convinced that the underlying mud was still of nearly the same liquidity as was shown by the tests (this was later confirmed during excavation);

(*e*) It would be impracticable, for some years, to provide a concrete floor in the hangar;

(*f*) The weight of the seaplane, upon its beaching gear, would be about 50 tons, which must be supported safely; and

(*g*) The foundation design should be such as to allow the water contained in the mud, and in the sand fill, to work its way out slowly. Some of the borings at the site are shown in Fig. 7.

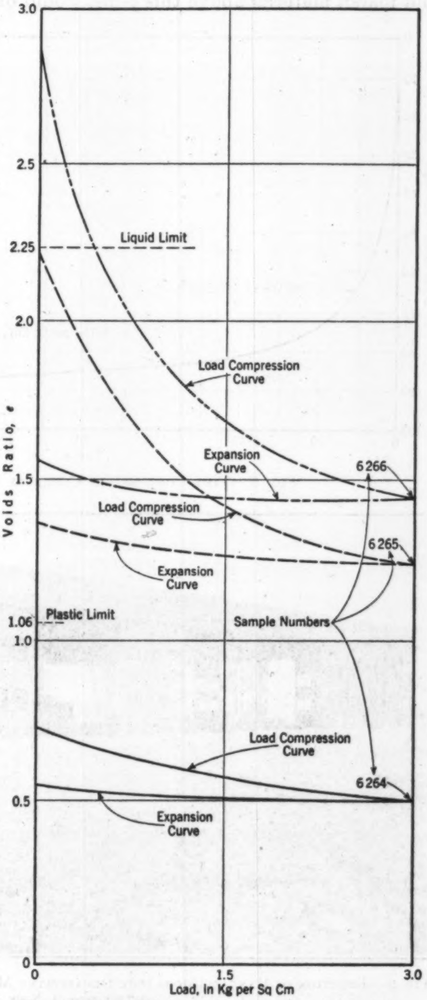


FIG. 4.—LOAD-COMPRESSION CURVES OF TWO MUD SAMPLES AND ONE SAND SAMPLE

THE PRINCIPLES UNDERLYING THE FOUNDATION DESIGN

The following conditions were established by the writer as the basis for the foundation design:

(1) There must be a minimum number of main column foundations;

(2) The major footings must be stable against a lateral thrust of 2,100 lb per sq ft, at the center, and 700 lb per sq ft, average, on the projected diameter;

(3) The mud pressure in an excavation must be considered as nearly that of a liquid weighing 100 to 120 lb per cu ft;

(4) Every effort must be made to prevent inflow of mud into the bottom of an excavation from the surrounding area; otherwise, the greater part of the subjacent mud would be excavated and the surface overlying it would sink;

(5) No lateral support for piling must be considered, except in the cylinders which were to be backfilled with granular fill;

(6) Piling must extend to −60.0 ft before meeting firm bottom, and the old marsh material above this point would offer resistance to penetration, but would provide a not-too-secure toe hold for the piling; hence, the piles must, if necessary, be jetted;

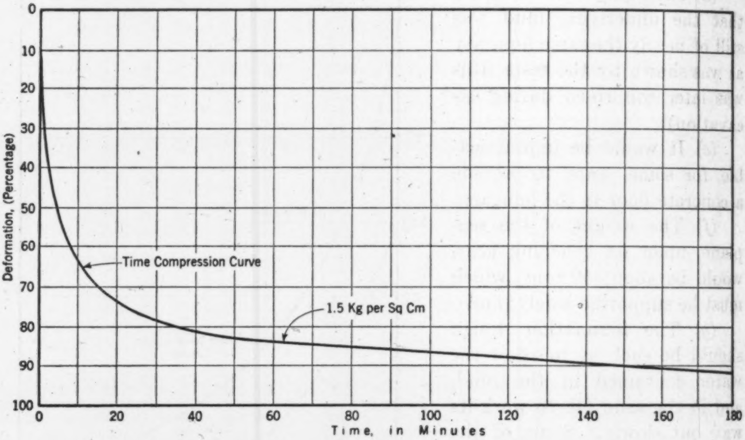


FIG. 5.—TIME-COMPRESSION CURVE OF ONE MUD SAMPLE (6265)

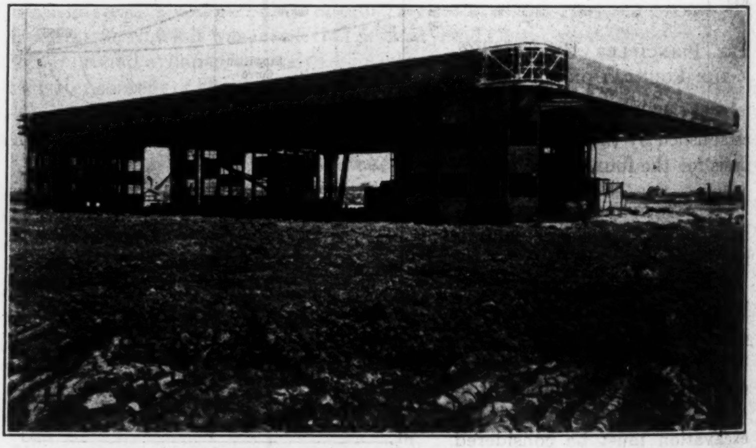


FIG. 6.—ERECTION OF HANGAR DOORS (THE DECAPITATED MUD WAVE IN THE FOREGROUND IS BEGINNING TO DEVELOP A NEW CRUST)

(7) The mud is a plastic—that is, it will stand shearing stress up to some (unknown) limit, but, beyond that stress, will flow like a viscous liquid, es-

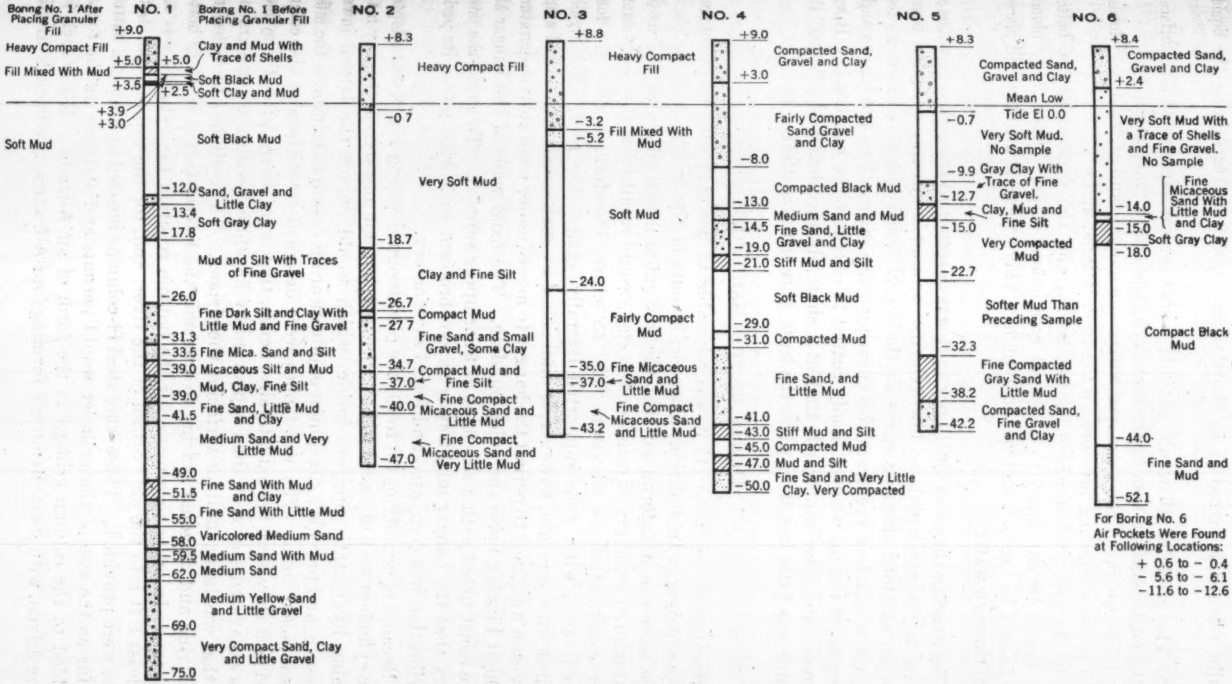

Boring No. 1 After Placing Granular Fill
Heavy Compact Fill
Fill Mixed With Mud
Soft Mud
+9.0
+5.0
+3.5
+3.9
+3.0
NO. 1
Boring No. 1 Before Placing Granular Fill
+5.0
+2.5
Clay and Mud With Trace of Shells
Soft Black Mud
Soft Clay and Mud
Soft Black Mud
-12.0
Sand, Gravel and Little Clay
-13.4
Soft Gray Clay
-17.8
Mud and Silt With Traces of Fine Gravel
-26.0
Fine Dark Silt and Clay With Little Mud and Fine Gravel
-31.3
-33.5
Fine Mica. Sand and Silt
-39.0
Micaceous Silt and Mud
-39.0
Mud, Clay, Fine Silt
-41.5
Fine Sand, Little Mud and Clay
Medium Sand and Very Little Mud
-49.0
Fine Sand With Mud and Clay
-51.5
-55.0
Fine Sand With Little Mud
-58.0
Varicolored Medium Sand
-59.5
Medium Sand With Mud
-62.0
Medium Sand
Medium Yellow Sand and Little Gravel
-69.0
Very Compact Sand, Clay and Little Gravel
-75.0
NO. 2
+8.3
Heavy Compact Fill
-0 7
Very Soft Mud
-18.7
Clay and Fine Silt
-26.7
Compact Mud
-27 7
Fine Sand and Small Gravel, Some Clay
-34.7
Compact Mud and Fine Silt
-37.0
Fine Compact Micaceous Sand and Little Mud
-40.0
Fine Compact Micaceous Sand and Very Little Mud
-47.0
NO. 3
+8.8
Heavy Compact Fill
-3.2
Fill Mixed With Mud
-5.2
Soft Mud
-24.0
Fairly Compact Mud
-35.0
Fine Micaceous Sand and Little Mud
-37.0
Fine Compact Micaceous Sand and Little Mud
-43.2
NO. 4
+9.0
Compacted Sand, Gravel and Clay
+3.0
Fairly Compacted Sand Gravel and Clay
-8.0
Compacted Black Mud
-13.0
Medium Sand and Mud
-14.5
Fine Sand, Little Gravel and Clay
-19.0
-21.0
Stiff Mud and Silt
Soft Black Mud
-29.0
-31.0
Compacted Mud
Fine Sand, and Little Mud
-41.0
-43.0
Stiff Mud and Silt
-45.0
Compacted Mud
-47.0
Mud and Silt
-50.0
Fine Sand and Very Little Clay. Very Compacted
NO. 5
+8.3
Compacted Sand, Gravel and Clay
Mean Low Tide El 0.0
-0.7
Very Soft Mud. No Sample
-9.9
Gray Clay With Trace of Fine Gravel.
-12.7
Clay, Mud and Fine Silt
-15.0
Very Compacted Mud
-22.7
Softer Mud Than Preceding Sample
-32.3
Fine Compacted Gray Sand With Little Mud
-38.2
Compacted Sand, Fine Gravel and Clay
-42.2
NO. 6
+8.4
Compacted Sand, Gravel and Clay
+2.4
Very Soft Mud With a Trace of Shells and Fine Gravel. No Sample
-14.0
Fine Micaceous Sand With Little Mud and Clay
-15.0
Soft Gray Clay
-18.0
Compact Black Mud
-44.0
Fine Sand and Mud
-52.1
For Boring No. 6
Air Pockets Were Found at Following Locations:
+ 0.6 to - 0.4
- 5.6 to - 6.1
-11.6 to -12.6

FIG. 7.—BORINGS AT THE TRANSATLANTIC SEAPLANE BASE

pecially when it is jarred as in the standard test for determining the liquid limit;

(8) The granular fill would provide ample lateral support at the surface, but the entire surface area in the vicinity might suffer translation;

(9) The weight of the seaplanes within the hangar, and of machinery loads, would cause a bursting pressure on the foundations; and

(10) As the mud slowly dries out below, and the surface settles, batter piles would be subjected to a lateral component stress; hence, only plumb piles should be used, except in the cylinders where there is granular fill placed down through water.

The average thrust of 700 lb per sq ft was determined from the equations of flow of a viscous liquid, at low velocity, around a circular cylinder, based upon an assumed shearing stress at the liquid limit; but there were no test data upon which to predicate this shearing value, other than the known angle of repose of the fill when it finally ceased to overflow the bulkheads. It was believed (and later confirmed) that at a short distance below the "crust" the fluidity was little less than when the tests were made in 1931.

Design of the Main Piers

Because of condition (1) it was advisable to use only four main columns for the hangar. In addition, however, layouts of the hangar and the planes to be stored in it determined that the maximum useful space, as well as the maximum of economy of construction cost, would result from the use of cantilever construction on all sides. The structure, therefore, is merely a four-legged table, with walls supported laterally against the roof. The rigidity against wind pressure, however, is diminished by such construction, and ample provision was made to permit the hangar to move (under wind and temperature) without binding upon the office building. The fixed walls of the hangar also were constructed with a slip joint at their upper support. To avoid the possibility that the hangar might sway in synchronism with wind gusts, its period of vibration was computed and found satisfactory.

Because of conditions (3) to (9) the pier design shown in Fig. 8 was adopted. The cylinders were designed to be excavated, but not unwatered, thereby maintaining hydrostatic pressure inside, which would reduce the lateral inward pressure, and provide a maximum of downward pressure to minimize the inflow of subjacent mud. The wood piles were designed to withstand the column load as if they were braced laterally; hence, they were driven after the cylinder was excavated and were then supported by backfilling with the same granular fill that was being used for surfacing the airport. The cylinders were designed with ring and diagonal bracing to transmit the lateral thrust to their bottom and top. In order to provide for the bottom reaction the cylinder was sunk at least 6 ft into the old bottom of the river, and, for the top reaction, batter piles were provided. These were steel H-columns incased in concrete, battered as far as the size of the cylinder would permit, and stripped at the top and welded to the column grillage in the form of an A-frame. Eight such piles were driven, with steam hammers, forming four A-frames, two in each direction.

They were not counted upon for vertical load, although of course they carry their share. The wind thrust on the hangar is also resisted by them.

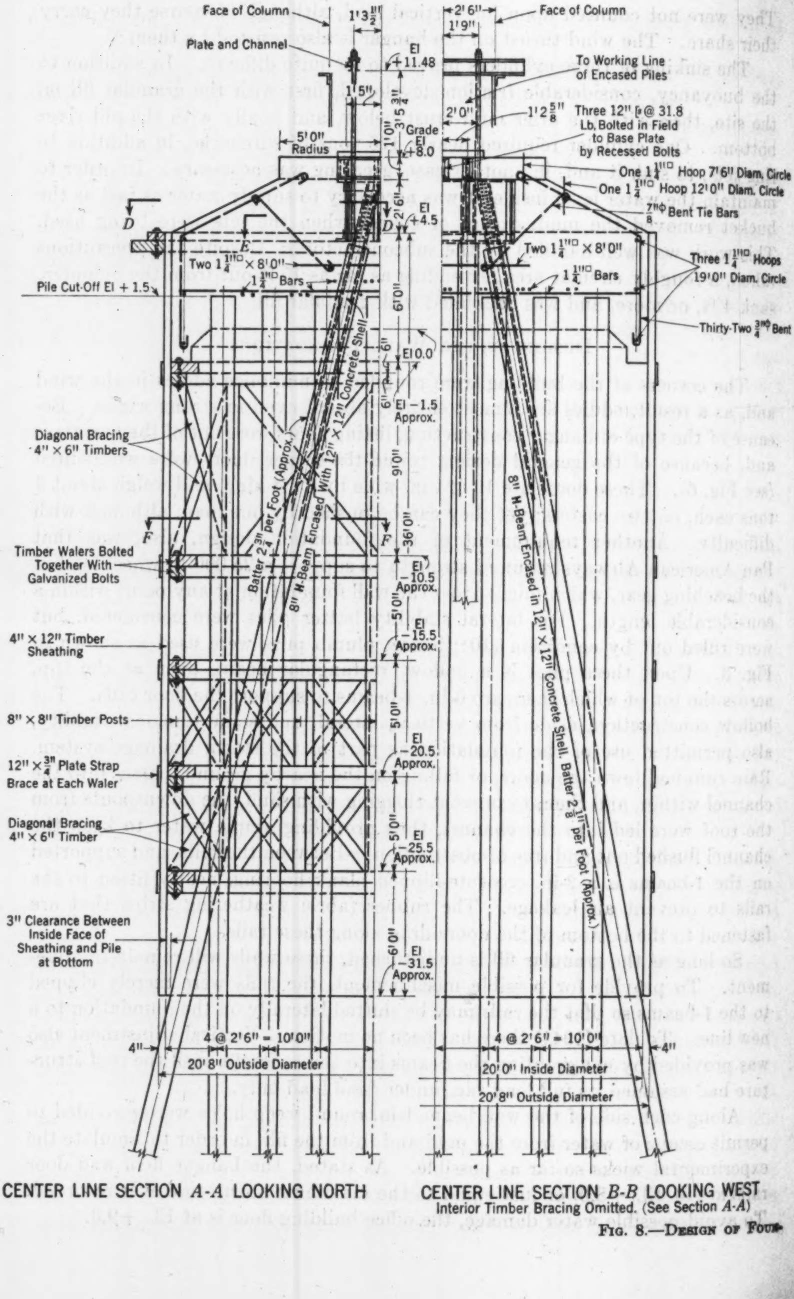
The sinking of these cylinders proved to be quite difficult. In addition to the buoyancy, considerable friction developed, first with the granular fill on the site, then with the drier mud crust below, and finally with the old river bottom. One cylinder required nearly 125 tons of surcharge, in addition to ring jets, to sink it and, in another case, greasing was necessary. In order to maintain the water level inside, it was necessary to supply water as fast as the bucket removed the mud, except of course when the jets were being used. This work was well handled by the subcontractors. Despite the precautions taken, a roughly circular area, extending as far as 15 ft out from the cylinder, sank 4 ft, or more, and was backfilled with granular fill.

DESIGN OF THE WALL FOUNDATIONS

The corners of the building were rounded to minimize eddies in the wind and, as a result, eddies are almost entirely absent even in strong winds. Because of the type of hangar construction, lifting doors were out of the question; and, because of the general design, round-the-corner doors were well suited (see Fig. 6). These doors are 10 ft 1 in. wide by 35 ft high, and weigh about 3 tons each, on two casters; yet they can be pushed by one man, although with difficulty. Another requirement of the foundation design, also, was that Pan American Airways required strength to support a 25,000-lb wheel load of the beaching gear, which might cross the wall foundation at any point within a considerable length. For lateral stability batter piles were considered, but were ruled out by condition (10); hence, plumb piles were used as shown in Fig. 3. Upon these piles is a hollow, rectangular beam, open at the top, across the top of which there are 6-in. I-beams to support the door rails. The hollow construction, aside from vertical, lateral, and some torsional rigidity, also permitted use of the foundation as part of the storm drainage system. Rain running down the doors, or falling on the nearby ground, passes into the channel within, and then, to prevent clogging with sand, the downspouts from the roof were led into the channel, thus providing ample water to keep the channel flushed out and free of obstruction. Between the rails, and supported on the I-beams is a 2-in. creosote-dipped plank flooring, neatly fitted to the rails to prevent air leakage. The rubber fabric weathering strips that are fastened to the bottom of the doors drag along these rails.

So long as the granular fill is undisturbed, these walls will remain in alinement. To provide for possible misalinement, the rails were merely clipped to the I-beams so that the rails may be shifted laterally on the foundation to a new line. To date (1941) there has been no motion. Vertical adjustment also was provided by not grouting the beams into the concrete until the roof structure had assumed its full load sag, under dead load only.

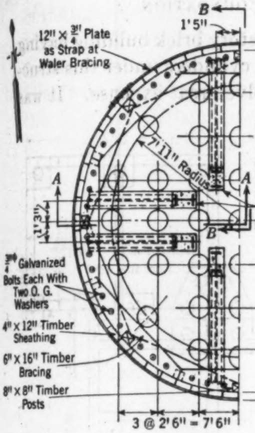
Along each side of the wall beam 1-in. round weep holes were provided to permit escape of water from the mud and from the fill, in order to simulate the experimental wicks so far as possible. As stated, the hangar floor and door rails are at El. +8.0, or just equal to the maximum storm tide ever recorded. To avoid possible water damage, the office building floor is at El. +9.0.


Face of Column
1' 3½"
Plate and Channel
El +11.48
1' 5"
3' 5¾"
1' 0"
Grade
El +8.0
5' 0" Radius
2' 6"
El +4.5
D
D
E
E
Two 1¼"□ × 8' 0"
1½"□ Bars
Pile Cut-Off El + 1.5
6' 0"
6"
El 0.0
6' 0"
El −1.5 Approx.
Diagonal Bracing 4" × 6" Timbers
9' 0"
Batter 2¾" per Foot (Approx.)
8" H-Beam Encased With 12" × 12" Concrete Shell
F
F
36' 0"
Timber Walers Bolted Together With Galvanized Bolts
El −10.5 Approx.
El −15.5 Approx.
4" × 12" Timber Sheathing
8" × 8" Timber Posts
5' 0"
12" × ¾" Plate Strap Brace at Each Waler
El −20.5 Approx.
Diagonal Bracing 4" × 6" Timber
5' 0"
El −25.5 Approx.
3" Clearance Between Inside Face of Sheathing and Pile at Bottom
6' 0"
El −31.5 Approx.
4"
4 @ 2' 6" = 10' 0"
20' 8" Outside Diameter
2' 6"
1' 9"
Face of Column
To Working Line of Encased Piles
2' 0"
1' 2⅝"
Three 12" [@ 31.8 Lb, Bolted in Field to Base Plate by Recessed Bolts
One 1¼"□ Hoop 7' 6" Diam. Circle
One 1¼"□ Hoop 12' 0" Diam. Circle
⅞"φ Bent Tie Bars
Two 1¼"□ × 8' 0"
Three 1¼"□ Hoops 19' 0" Diam. Circle
1¼"□ Bars
Thirty-Two ¾"φ Bent
8" H-Beam Encased in 12" × 12" Concrete Shell. Batter 2⅝" per Foot (Approx.)
4 @ 2' 6" = 10' 0"
4"
20' 0" Inside Diameter
20' 8" Outside Diameter

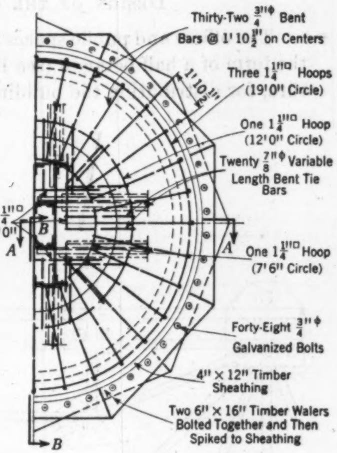
CENTER LINE SECTION *A-A* LOOKING NORTH

CENTER LINE SECTION *B-B* LOOKING WEST
Interior Timber Bracing Omitted. (See Section *A-A*)

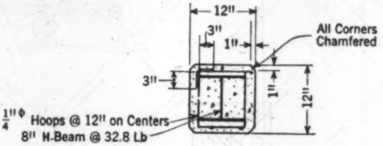
Fig. 8.—Design of Four


12" × 3/4" Plate as Strap at Waler Bracing
B
1'5"
7'11" Radius
8" H-Beam Encased in 12" × 12" Concrete
A
1'3"
B
3/4" Galvanized Bolts Each With Two O. G. Washers
4" × 12" Timber Sheathing
6" × 16" Timber Bracing
8" × 8" Timber Posts
6 @ 2'6" = 15'0"
20'0" Inside Diameter
20'8" Outside Diameter
3 @ 2'6" = 7'6"

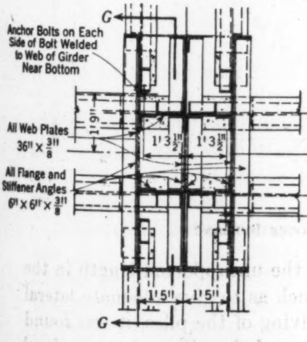
SECTION *F-F*


Thirty-Two 3/4"ϕ Bent Bars @ 1' 10 1/2" on Centers
1' 10 1/2"
Three 1 1/4"□ Hoops (19'0" Circle)
One 1 1/4"□ Hoop (12'0" Circle)
Twenty 7/8"ϕ Variable Length Bent Tie Bars
Four 1 1/4"□ × 8'0"
A
B
One 1 1/4"□ Hoop (7'6" Circle)
Forty-Eight 3/4"ϕ Galvanized Bolts
4" × 12" Timber Sheathing
Two 6" × 16" Timber Walers Bolted Together and Then Spiked to Sheathing
B

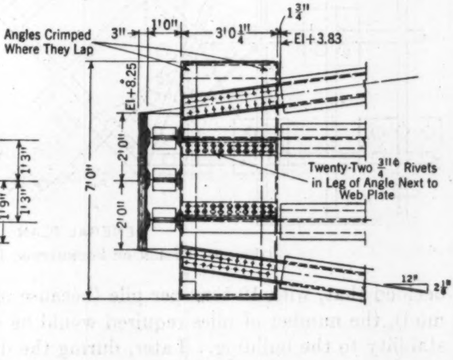
SECTION *D-D*


12"
3"
1"
All Corners Chamfered
3"
1"
12"
1/4"ϕ Hoops @ 12" on Centers
8" H-Beam @ 32.8 Lb

SECTION THROUGH CONCRETE PILES


G
Anchor Bolts on Each Side of Bolt Welded to Web of Girder Near Bottom
All Web Plates 36" × 3/8"
All Flange and Stiffener Angles 6" × 6" × 3/8"
1'9"
1' 3 1/2"
1' 3 1/2"
1'3"
1'9"
1'3"
1'5"
1'5"
G

SECTION *E-E*


Angles Crimped Where They Lap
3"
1'0"
3' 0 1/4"
1 3/4"
El+3.83
El+8.25
7'0"
2'0"
2'0"
Twenty-Two 3/4"ϕ Rivets in Leg of Angle Next to Web Plate
12"
2 1/8"

SECTION *G-G*

DATIONS FOR MAIN COLUMNS

Design of the Office Building Foundation

The offices and public spaces are located in a two-story brick building having the form of a half octagon (see Fig. 9). To provide cylinders under this structure, for distributing the building loads, would involve great expense. It was decided that, with 10 tons per pile (because of the unsupported length in the mud), the number of piles required would be such as to give adequate lateral stability to the building. Later, during the driving of the piles, it was found that penetration could not reach −60, and some of the piles came to refusal at only 52 to 54 ft, and one or more at 48 ft. Therefore, the toe-hold capacity

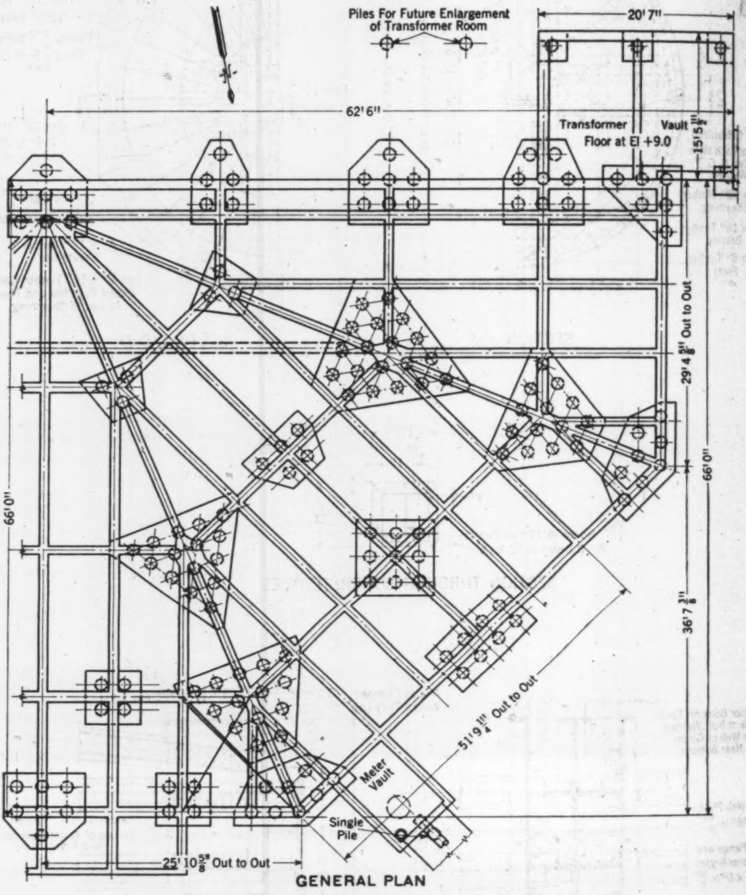

Piles For Future Enlargement
of Transformer Room
20'7"
62'6"
Transformer
Vault
Floor at El +9.0
15'5½"
29'4 5/8" Out to Out
66'0"
66'0"
36'7 3/8"
51'9 1/4" Out to Out
Meter Vault
Single Pile
25'10 5/8" Out to Out
GENERAL PLAN

Fig. 9.—Design of Foundation, Office Building

of the old river bottom is at least as good as was assumed: It was impracticable to use the jet on these piles, because the area under the building was excavated to El. +5.0 (for an air space under the first floor), which space would have filled with water and drowned out all nearby work. At several footings under this building the underlying mud was exposed, and some of the excavations for them filled with mud which was exuded up through the bottom.

Design of the Hangar

There were no precedents for the design of the hangar. The Pan American Airways System stipulated the over-all dimensions and the clear height as 270 ft by 190 ft by 35 ft. They desired two rows of columns, giving two cantilever ends. The further restriction to four columns only permitted cantilever construction on all four sides, and also the saving that results in truss weight. Larger planes can be placed within, also, such that the trailing edge of the wing just touches the columns. The span of the wings thus can be greater than the column spacing; also, by entering askew, only one wing tip need clear a column (see Figs. 10, 11, and 12).

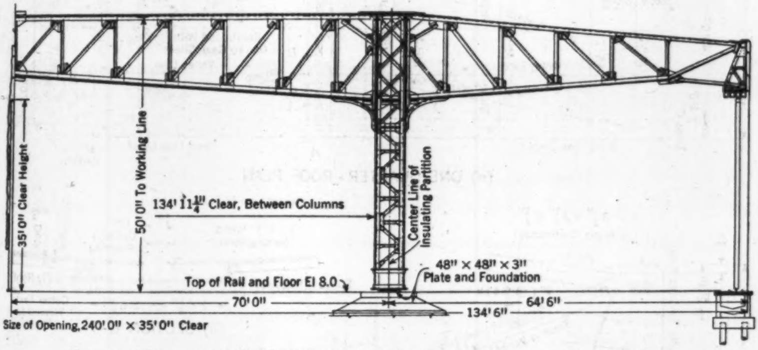

35' 0" Clear Height
50' 0" To Working Line
134' 11¼" Clear, Between Columns
Center Line of Insulating Partition
48" × 48" × 3" Plate and Foundation
Top of Rail and Floor El 8.0
70' 0"
64' 6"
134' 6"
Size of Opening, 240' 0" × 35' 0" Clear

Fig. 10.—Design of Seaplane Hangar; Half-Section Showing Typical Truss

The unit load specifications,[3] in pounds per square foot, are as follows: Snow load, 30; roof uplift, 40; and wall loads, 40 on small areas, 30 on each door or wall panel, and 20 on the entire building. The 20-lb load acts at 135° to the larger wall, with wind also on the adjacent side, and the resultant wind acts at an eccentricity from the center of the building of 8% of the wall length, thus causing rotation.

The total wind was divided equally between the four columns, which were computed as though hinged at the base. (The torsion, also, was equally distributed.) They are 5 ft square, and bear on 24-in. steel sole slabs, which in turn bear on 48-in. slabs welded to the grillage. This 48-in. size was adopted to provide for tolerance in setting the grillage. However, the maximum

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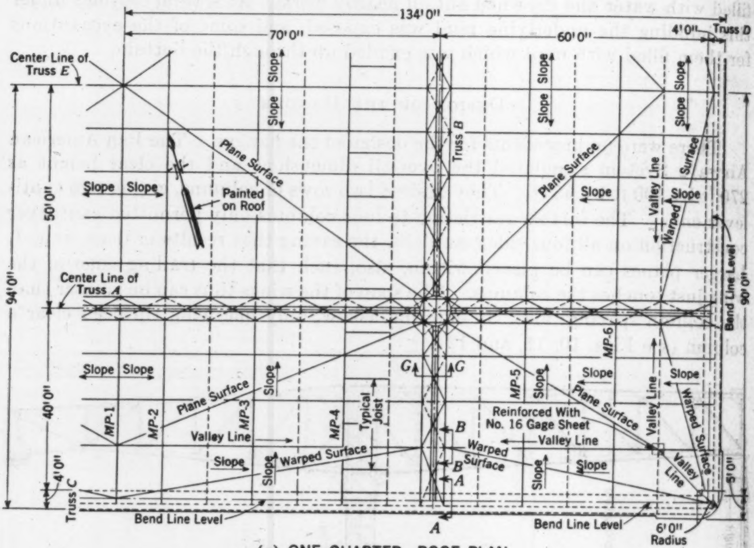
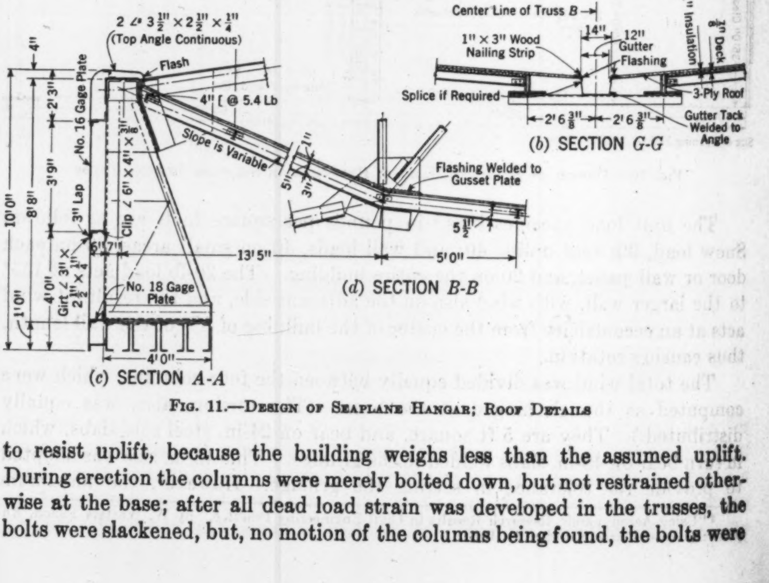
[3] "Using Aerodynamic Research Results in Civil Engineering Practice," by W. Watters Pagon, *Engineering News-Record*, October 31, 1935.

uncorrected error in the grillage positions was between 1 in. and 2 in. The trusses were computed for the column bending moment, and then curved knee-braces were added. There are two 3-in. anchor bolts, welded to the grillage,


134'0"
70'0"
60'0"
4'0"
Truss D
Center Line of Truss E
Slope
Plane Surface
Painted on Roof
Truss B
Valley Line
Warped
Bend Line Level
50'0"
94'0"
90'0"
Center Line of Truss A
MP-1
MP-2
MP-3
MP-4
MP-5
MP-6
Typical Joist
G
Reinforced With No. 16 Gage Sheet
Valley Line
Warped Surface
B
A
40'0"
4'0"
5'0"
Truss C
Bend Line Level
6'0" Radius

(a) ONE QUARTER - ROOF PLAN


Center Line of Truss B
1" × 3" Wood Nailing Strip
14"
12" Gutter Flashing
1" Insulation
1/8" Deck
Splice if Required
3-Ply Roof
2'6 3/8"
2'6 3/8"
Gutter Tack Welded to Angle
2 ∠s 3 1/2" × 2 1/2" × 1/4" (Top Angle Continuous)
Flash
4"
2'3"
No. 16 Gage Plate
4" [@ 5.4 Lb
Slope is Variable
3'9"
8'8"
10'0"
Clip ∠ 6" × 4" × 3/16"
3" Lap
1"
5"
3"
Flashing Welded to Gusset Plate
5 1/2"
6"
7"
13'5"
5'0"
4'0"
Girt ∠ 3" × 2 1/2" × 1/4"
No. 18 Gage Plate
1'0"
4'0"

(b) SECTION G-G

(c) SECTION A-A

(d) SECTION B-B

FIG. 11.—DESIGN OF SEAPLANE HANGAR; ROOF DETAILS

to resist uplift, because the building weighs less than the assumed uplift. During erection the columns were merely bolted down, but not restrained otherwise at the base; after all dead load strain was developed in the trusses, the bolts were slackened, but, no motion of the columns being found, the bolts were

again tightened and a 3-in. angle was welded to the bottom slab on all four sides to prevent sliding.

The upper tracks for the doors consist of 12-in. channels, facing each other, which are also computed as part of the chord section of the fascia trusses. These trusses are about 9 ft deep, of triangular section, and are braced by knees to the purlins to prevent torsion, although the vertical load is near the centroid. There is a 48-in. wide bottom plate, to which the four door tracks are riveted, and this plate and the channels form a horizontal girder to resist the wind thrust on the doors and on the fascia, in addition to acting as bottom chord (see Figs. 11 and 13).

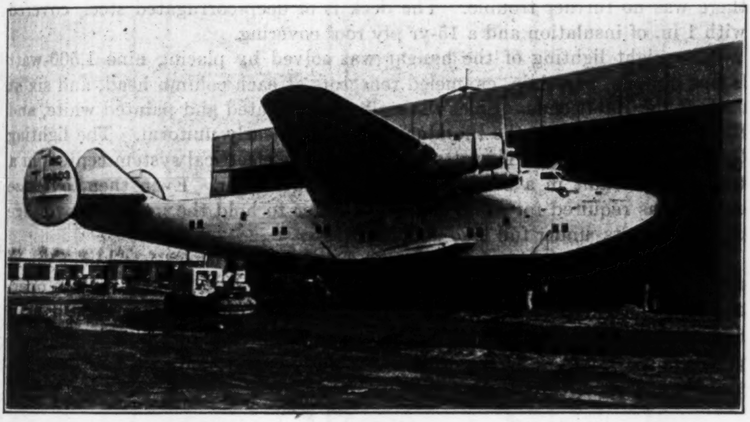


FIG. 12.—YANKEE CLIPPER ENTERING WEST DOOR OF HANGAR, FEBRUARY 26, 1939

The general structural design is shown in Figs. 10 and 11. When completed, the structure is indeterminate. It was computed as though determinate (with later corrections for continuity) and was so erected. The fascia trusses were carefully cambered so as to be truly straight and horizontal under full dead load, and the end connections between them, at the four corners of the hangar, were not installed until all dead load was in place. Continuity in the purlin knee-braces, however, prevented perfect vertical alinement of these fascia trusses, and a minor correction was necessary in the elevation of the bottom I-beams (which had not been grouted into the foundation) to make up for the discrepancy.

The end panel of the cantilevers was cambered upward for full dead load. Because of the cantilever construction there is much sag, or lift, in the fascia trusses. At the top of the doors provision was made for a sag, or a lift, of 3 in., due to snow or uplift (the computed distance was 2½ in.). For plumbing the doors an adjustment of ¾ in. was made in each caster. The doors and hardware are larger than any standard design; hence, they both were computed and designed especially for this structure (see Fig. 13).

The roof structure is unusual. For three reasons it is inverted, with the roof supported upon the bottom chord of the main trusses and purlins. The first reason (historically) was roof drainage, there being no place for downspouts except at the four columns. (The building corners are so designed that they may be put on casters later, if future wing spans require greater useful opening.) The other, and more determining reasons, are the protection of the structure from inside fires, and the reduction in the height of the space to be heated. A ceiling would have added a considerable percentage to the total load. The roof-deck design offered many problems, but they were successfully met as shown in Fig. 11. At first there were a few leaks in the flashing collars around the gusset plates of trusses and purlins, but when these had been corrected there was no further trouble. The deck is of deep-corrugated steel, covered with 1 in. of insulation and a 15-yr ply roof covering.

The night lighting of the hangar was solved by placing nine 1,500-watt lamps in open, porcelain, enameled reflectors at each column head, and six at each of the 190-ft ends. Since the walls are insulated and painted white, and the roof is galvanized, the illumination is very nearly uniform. The lighting load is 72 kw, or about 1½ watts per sq ft. The electrical system centers in a transformer vault in a brick room within the hangar. Even then, oversize copper was required for some circuits in order to hold the maximum voltage drop to 5 volts, under full load conditions.

In order to eliminate piping runs under the floor, where settlement might cause breaks, all steam and return piping is at the ceiling, and the system operates at sufficient pressure to lift the condensate from the floor type of unit heaters and return it to the boilers. The floor is a bituminous concrete, placed on a sand fill which was thoroughly wetted and rolled with a steam roller, and has given perfect satisfaction.

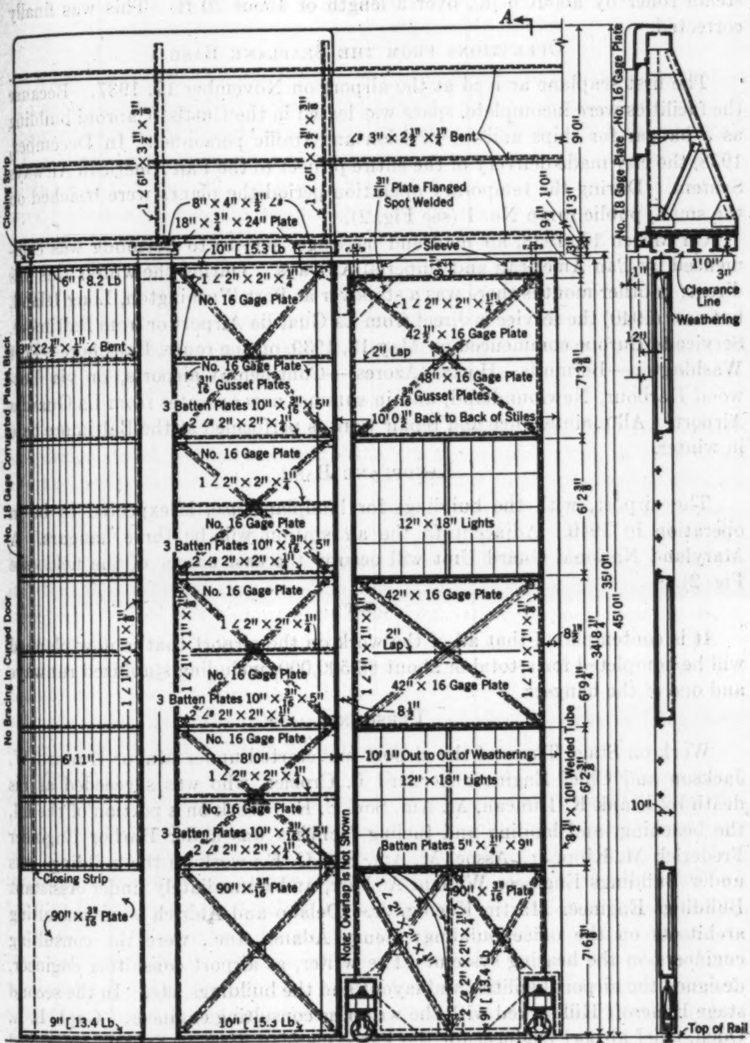
DESIGN OF THE OFFICE BUILDING

The office building houses all the operating and traffic personnel, and also a portion of the division engineering personnel (the remainder are in small shop offices in the hangar). There is space for tickets, facing the Concourse, and on the other side are rooms, in series, for Public Health and Immigration inspection. In the rear of the Concourse is the customs inspection counter.

LANDING, BEACHING, AND FUELING FACILITIES

Passengers are landed at a float and pass over a floating walkway and then a pile trestle to a covered walkway. This walkway leads directly into a waiting room at the entrance to the federal quarters. The pump house at the end of the walkway provides gasoline either on the float or at the head of the beaching ramp. There is underground storage for 16,000 gal of gasoline.

Shortly before completion of the hangar it was decided, as an additional precaution, to install a standard-gage railroad track leading into the south and the west doors. No trouble was experienced, however, except at the mud wave south of the hangar, where the track deflected under the weight of a 10-ton


A
No. 18 Gage Corrugated Plates, Black
Closing Strip
No Bracing in Curved Door
6" [8.2 Lb
3"×2½"×¼" ∠ Bent
1 ∠ 1"×1"×⅛"
6' 11"
Closing Strip
90"×3/16" Plate
9" [13.4 Lb
1 ∠ 6"×3½"×⅜"
8"×4"×½" ∠s
18"×¾"×24" Plate
10" [15.3 Lb
1 ∠ 2"×2"×¼"
No. 16 Gage Plate
3/16" Gusset Plates
3 Batten Plates 10"×3/16"×5"
2 ∠s 2"×2"×¼"
8' 0"
90"×3/16" Plate
10" [15.3 Lb
Note: Overlap is Not Shown
∠s 3"×2½"×¼" Bent
3/16" Plate Flanged Spot Welded
Sleeve
42"×16 Gage Plate
2" Lap
48"×16 Gage Plate
3/16" Gusset Plates
10' 0¼" Back to Back of Stiles
12"×18" Lights
8½"
10' 1" Out to Out of Weathering
Batten Plates 5"×¼"×9"
1 ∠ 2"×2"×¼"
9" [13.4 Lb
A
No. 16 Gage Plate
No. 18 Gage Plate
1' 0"
3"
Clearance Line
Weathering
9' 0"
1' 0"
1' 3"
9½"
12"
7' 3⅞"
6' 2⅜"
35' 0"
45' 0"
34' 8¼"
6' 9¼"
6' 2⅜"
7' 6⅜"
8½"×10" Welded Tube
10"
3¼"
Top of Rail

(a) ELEVATION CURVED PANEL (b) ELEVATION 8' 0" PANEL (c) ELEVATION 10' 0¼" DOOR (d) SECTION A-A

FIG. 13.—DESIGN OF DOORS

steam roller by about 6 in., over a length of about 20 ft. This was finally corrected.

OPERATIONS FROM THE SEAPLANE BASE

The first seaplane arrived at the airport on November 17, 1937. Because the facilities were incomplete, space was leased in the Curtiss-Caproni building as a hangar for ships and for division and traffic personnel. In December, 1938, the city made delivery of the entire project to the Pan American Airways System. During the temporary operation period the planes were beached on the small, public ramp No. 1 (see Fig. 2).

On March 16, 1938, air mail and passenger service to Bermuda was commenced by Pan American and Imperial Airways. During the winter this was direct; in other months there was a stopover at Port Washington, Long Island, but now (1940) the service is direct from La Guardia Airport or from Baltimore. Service to Europe commenced on May 13, 1939, on the route, Baltimore—Port Washington—Bermuda—Horta (Azores)—Continental airports, or via Botwood Harbour, Newfoundland, and in summer now operates from La Guardia Airport. All maintenance and repair work is performed at the Baltimore base in winter.

LANDPLANE BASE

The airport, with the buildings for landplane use, is expected to be in operation in 1940. Adjacent to the air station will be three hangars. A Maryland National Guard Unit will occupy the east corner of the field (see Fig. 2).

COST

It is contemplated that all of the work on the airport that is now planned will be completed for a total of about $6,500,000, including stabilized runways and one of the hangars.

PERSONNEL

Work on Stage Three of the airport was started under Mayor Howard W. Jackson and Chief Engineer Bernard L. Crozier, who was succeeded at his death by Frank K. Duncan, M. Am. Soc. C. E. Work on a portion of the fill, the beaching, and landing and fueling facilities was under Harbor Engineer Frederick M. Kipp, Jr., Assoc. M. Am. Soc. C. E.; work on the buildings was under Buildings Engineer William A. Parr, and immediately under Assistant Buildings Engineer Martin Koenig, Jr. Delano and Aldrich were consulting architects on the office building; Henry Adams, Inc., were the consulting engineers on the heating system. The writer, as airport consulting engineer, designed the airport facilities and layout and the buildings, etc. In the second stage Bancroft Hill served with the writer as consulting engineer. Capt. L. L. Odell, chief airport engineer for the Pan American Airways System, approved the plans for the company. Henry L. Shryock, Jr., was project engineer for the PWA. The contractors were as follows: For the fill, The Arundel Corporation; for foundations, Consolidated Engineering Company, under whom was the Raymond Concrete Pile Company on the main piers and the piling; for the hangar, Kaufman Construction Company; and for the office building, W. E. Bickerton Construction Company.

DISCUSSION

Karl Terzaghi,[4] **M. Am. Soc. C. E.** (by letter).—During the construction of the seaplane base at Baltimore, the author had an unusual opportunity to observe the gradual consolidation of an hydraulic mud fill. Under the heading "The Mud" he mentions that he used wicks for the purpose of accelerating the drainage of the mud fill. A brief statement of the mode of application of the wick principle, and of the results obtained by means of this method, would certainly be of general interest. The hangar floor consists of a bituminous concrete placed on a rolled sand fill. The sand fill seems to be supported by a granular fill which in turn rests on an incompletely consolidated mud fill. The writer would appreciate some information on the rate at which the floor settles and on the method which is used for the periodic readjustment of its position, provided such readjustment is required.

LeRoy L. Odell,[5] **Esq.** (by letter).—The basic conceptions from which this design developed became established in the writer's mind in 1933 in an effort to perfect an economical form of hangar that would possess the greatest possibility of future utility, having in mind the recurring step-by-step increases in the size of aircraft, which still continue to render obsolescent most earlier hangar buildings. An indication of such a need will become apparent from the fact that it is believed that first-line land aircraft in domestic commercial air transport services will have capacities approximating 80 passengers by 1950, in comparison with 33-passenger ships—today's largest. Mr. Pagon's reduction of the number of columns to four, enforced by foundation conditions, is a definite improvement upon the original plans from which his design was developed, which had a cantilever roof structure over two rows of irregularly spaced columns, and doors on all four sides.

Another matter of interest in the design features of the Baltimore hangar is the fact that wind loadings are assumed as being negative (uplift) upon the roof. Design practices that contemplate the resolution of wind forces into components parallel and normal (pressure) to roof lines are erroneous. Wind tunnel experiments demonstrate clearly that such method is incorrect and, as early as 1929, the writer, as chief airport engineer for Pan American Airways, began the control of hangar design upon the basis that wind loadings upon roofs were negative (uplift). No Pan American Airways' hangar has yet suffered wind storm damage (except that caused by flying debris from other sources) notwithstanding the fact that several such hangars have passed through hurricanes in which literally thousands of lives have been lost.

Architects and engineers responsible for the structural design of buildings would do well to look carefully into this factor in their design problems, since thereby substantial savings, together with added safety, become assured.

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[4] Dr. Ing., Lecturer, Harvard Univ., Cambridge, Mass.

[5] Engr., New York, N. Y.

W. WATTERS PAGON,[6] M. AM. SOC. C. E. (by letter).—Professor Terzaghi asks two questions, which are interrelated. Wicks, as such, could not be used, but the principle involved was applied along the foundation that supports the rolling doors and the fixed walls. A part of this foundation, at the southwest corner, was located at a point where the depth of the granular fill was deficient; hence, some means of consolidating the mud below would stiffen the mud and create additional bearing power. Again, this foundation along its entire length was designed for the heavy concentrated loads of the seaplane and its beaching gear, whereas the granular fill that supports the floor has no such bearing power; hence, a means of drying the mud along this foundation is effective in bringing about more uniformity along the line where the beaching gear passes from the rigid foundation to the resilient fill. The means adopted was the insertion of frequent 1-in. holes through the side of the foundation to permit flow of water from the mud, through the granular fill, into the waterway located inside of the foundation. The result has been that there is a total settlement at the southeast corner of about 6 in., and at sporadic points along the perimeter there are similar settlements of from 2 in. to 4 in.

The settlement of the floor in general cannot be determined with accuracy, because the finished surface of the bituminous concrete floor covering was not accurate to within perhaps 2 in., due to the fact that it was put down and rolled with a steam roller. The approximate settlement varies from 0 to about 4 in., and the present contour is a gently rolling surface. Along one line, for a reason not yet determined, there is a settlement of about 3 in. to 4 in., forming in cross section an ogee type of curve. At one short length of the beaching railroad track there is a sag of about 2 in., but it is interesting to note that there is no evidence of settlement at the two points, one on each track, where the seaplanes and their gear stood for days at a time during overhaul. At one of these spots a seaplane stood continuously for about two months while repairs were being made.

Captain Odell very properly emphasizes the specification for uplift on the roof due to wind. Not only do tests show considerable suction, but the writer has repeatedly observed the eddy that covers about one half of the windward part of the roof, and the space within an eddy is under lower pressure than the streamline flow. In the design of hangars consideration must be given to the chance that a sudden thundersquall may develop at a time when there is a considerable area of open doors. Elsewhere,[7] the writer has discussed the pressure developed within a building when there are only small aperatures on the windward side, whereas the open doors of a hangar will permit an inside pressure of about 20 lb per sq ft when the velocity head of the wind velocity amounts to 30 lb per sq ft. With an inside pressure of 20 and an outside suction of 20 lb per sq ft, there may be a total outward force of 40 lb per sq ft over large areas.

The writer is indebted to the discussers for the thought which they have given to the paper, and is appreciative of their views.

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[6] Cons. Engr., Baltimore, Md.

[7] "Wind Tunnel Studies Reveal Pressure Distribution on Buildings," by W. Watters Pagon, *Engineering News-Record*, December 27, 1934.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 2123

EARTHQUAKE STRESSES IN THE SAN FRANCISCO-OAKLAND BAY BRIDGE

By Norman C. Raab,[1] M. Am. Soc. C. E., and Howard C. Wood,[2] Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Maurice A. Biot, Franklin P. Ulrich, Leon S. Moisseiff, and Norman C. Raab and Howard C. Wood.

Synopsis

The combination of deep piers and long spans, of various types and degrees of rigidity, in the San Francisco-Oakland Bay Bridge, resulted in unusual problems in the calculation of earthquake stresses. This paper describes the assumptions which were made, typical examples of stress analysis, and the provisions incorporated to resist seismic forces. It was found that an acceleration equivalent to 10% of gravity produced stresses in the superstructure of the same order as those arising from the assumed wind loads. In general, very little additional material was required to resist earthquake forces. In the following, use has been made of data, including vibration measurements on the structure, not available at the time of the actual design. To such extent, this paper may be considered a review of the design.

Description of Bridge

The bridge, with its approaches, extends from San Francisco, Calif., to Oakland, Calif., a distance of 8½ miles. It is naturally divided into the San Francisco Section, the West Bay Crossing, the Island Crossing, the East Bay Crossing, and the East Bay Approaches. The structures on the approaches and on the Island were such as to permit usual methods of seismic analysis and will receive no further comment herein.

The West Bay Crossing extends from the San Francisco Anchorage to Yerba Buena Island (see Fig. 1(*a*)) in San Francisco Bay. It consists of a twin suspension structure with a common central anchorage. The center spans are each 2,310 ft, and the side spans are each 1,160 ft in length. To secure better foundations, the San Francisco Anchorage was located 863 ft from the end of the suspended structure. This resulted in a comparatively long backstay cable which makes the west half of the suspension structure somewhat more

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Note.—Published in October, 1940, *Proceedings*.

[1] Senior Bridge Engr., State Dept. of Public Works, Sacramento, Calif.

[2] Senior Bridge Engr., San Francisco-Oakland Bay Bridge, Oakland, Calif.

flexible than the east half. The piers range in depth from 100 to 240 ft. At Pier 6, the mud line is 105 ft below low water. The strata through which the piers extend consist of about 20 ft of comparatively soft silt, underlain by consolidated clays and sands. All piers are founded on rock (Franciscan sandstone).

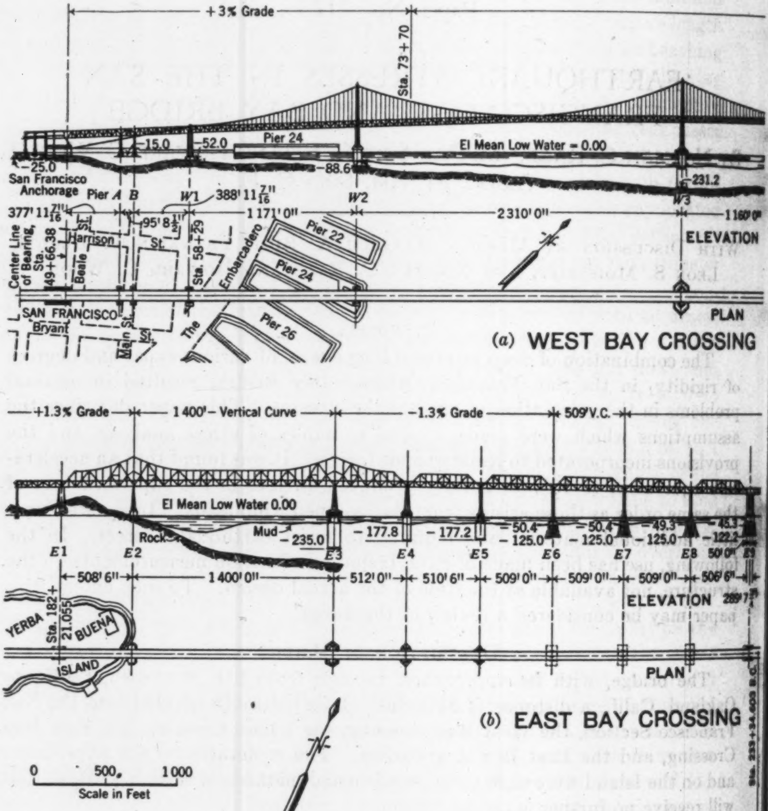
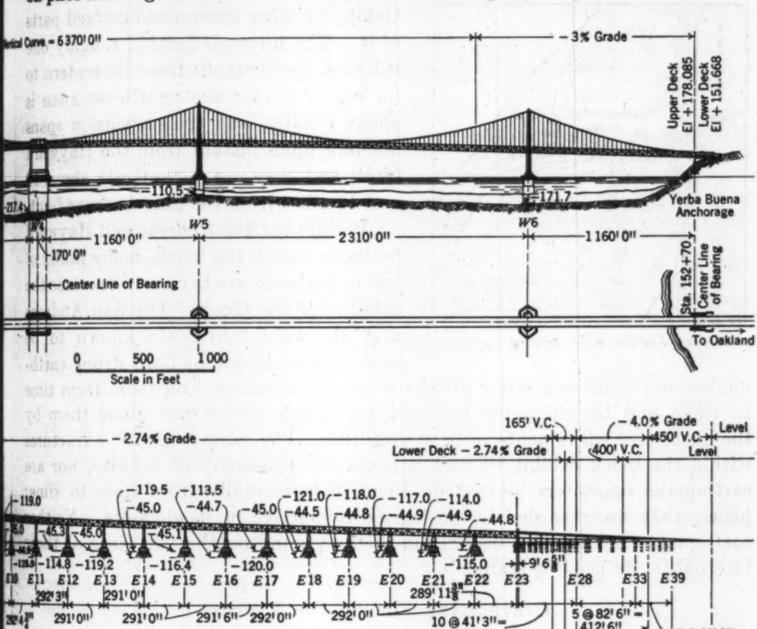

+3% Grade
Sta. 73+70
Pier 24
–15.0
–52.0
El Mean Low Water = 0.00
–25.0
–88.6
–231.2
San Francisco Anchorage
Pier A
B
W1
W2
W3
377' 11 7/16"
388' 11 7/16"
95' 8 1/2"
1171' 0"
2310' 0"
1160' 0"
ELEVATION
Center Line of Bearing, Sta. 49+66.38
Harrison St.
Beale St.
Main St.
Bryant St.
Sta. 58+29
The Embarcadero
Pier 22
Pier 24
Pier 26
SAN FRANCISCO
PLAN
(a) WEST BAY CROSSING
+1.3% Grade
1400' – Vertical Curve
–1.3% Grade
509' V.C.
El Mean Low Water 0.00
Mud
Rock
–235.0
–177.8
–177.2
–50.4
–125.0
–50.4
–125.0
–49.3
–125.0
–45.3
–122.2
E1
E2
E3
E4
E5
E6
E7
E8
E9
508' 6"
1400' 0"
512' 0"
510' 6"
509' 0"
509' 0"
509' 0"
506' 6"
ELEVATION
YERBA BUENA ISLAND
Sta. 182+21.055
PLAN
(b) EAST BAY CROSSING
0
500
1000
Scale in Feet

FIG. 1.—PLAN AND ELEVATION,

The East Bay Crossing (Fig. 1(*b*)) extends from the Island to the Oakland shore line. It consists of four 288-ft spans, a cantilever structure with a 1,400-ft center span, and two 508-ft anchor arms, five 504-ft and fourteen 288-ft spans.

To provide anchorage against longitudinal forces, the structure is anchored at piers YB-1, E-1, E-9, and E-17, with provision for expansion at piers YB-3, E-4, and E-11. At Pier E-4, provision is made for the expansion, arising from temperature and stress, of nearly a mile of bridge. East of the Island, the rock slopes off very sharply. Pier E-2 is founded on rock at El. −40. Pier E-3 extends to a sand and gravel stratum at El. −240, and piers E-4 and E-5 are founded on similar strata at El. −170. The remaining piers were supported on piles although their bases were extended to comparatively solid sand strata.


rtical Curve = 6 370' 0"
3% Grade
Upper Deck El + 178.085
Lower Deck El + 151.668
110.5
171.7
Yerba Buena Anchorage
W4
W5
W6
1 160' 0"
2 310' 0"
1 160' 0"
170' 0"
Center Line of Bearing
Sta. 152+70
Center Line of Bearing
To Oakland
0
500
1 000
Scale in Feet
165' V.C.
− 4.0% Grade
Level
− 2.74% Grade
Lower Deck − 2.74% Grade
400' V.C.
450' V.C.
Level
−119.0
−119.5
−113.5
−45.0
−44.7
−121.0
−118.0
−117.0
−114.0
−45.0
−44.5
−44.8
−44.9
−44.9
−44.8
−45.3
−45.0
−45.1
−114.8
−119.2
−116.4
−120.0
−115.0
9' 6 5/8"
E10
E11
E12
E13
E14
E15
E16
E17
E18
E19
E20
E21
E22
E23
E28
E33
E39
292' 3"
291' 0"
289' 11 3/8"
291' 0"
291' 0"
291' 6"
292' 0"
292' 0"
10 @ 41' 3" = 412' 6"
5 @ 82' 6" = 412' 6"
6 @ 41' 3" = 247' 6"
Sta. 275+72.347 B.V.C.
Sta. 276+48.140 B.V.C.
Sta. 277+37.347 B.V.C.
Sta. 280+48.140 E.V.C.
Sta. 282+30.997 B.V.C.
Sta. 283+97.347
Sta. 286+80.997 E.V.C.
End of Upper Deck Structure
Sta. 273+15.295
Center Line of Bearing
Curve Data
Δ = 10° 55' 13.63"
R = 2000'
T = 191.177
L = 381.196'
Sta. 257+15.799 E.C.
Sta. 253+24.603 B.C.
KEY SYSTEM FILL
To Oakland

West Bay and East Bay Crossings

GEOLOGY

San Francisco Bay lies on the surface of a long fault block or strip of the earth's crust trending roughly northwest and southeast (see Fig. 2). The block is bounded on the east by the Hayward fault, a nearly vertical fracture located along the west base of the Berkeley Hills and hence along the eastern margin of such East Bay cities as Berkeley and Oakland, Calif. Along the west side of the block is the San Andreas fault, likewise a vertical break in the crust, and lying a few miles offshore from the mouth of the Golden Gate. The block is hence about 20 miles wide, its length is many times its width, and its depth is not less than 25 or 30 miles. The Bay only occupies a fraction of its width, the cities of San Francisco and Oakland resting upon unsubmerged parts of it. The bridge is located roughly one third of the distance from its eastern to its western side; the cantilever span is about 7 miles, and the suspension spans about 9 miles distant from the Hayward fault, and they are respectively about 12 and 10 miles from the San Andreas fault.

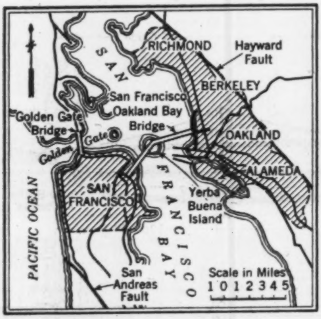


Fig. 2.—Location of Active Faults

Besides the San Andreas and Hayward faults bounding the block, many faults of less importance are known to traverse the interior of the block. The San Andreas and Hayward faults are known to be active, as evidenced by both strong earthquakes and numerous minor shocks which have occurred on them from time to time, and by characteristic fault topography developed along them by the successive movements in past centuries. The numerous minor fractures within the block exhibit no such topographic testimony of activity, nor are earthquake epicenters located on them instrumentally from time to time; hence, they are regarded by geologists as inactive or dead faults. Neither earthquakes nor relative dislocation of the ground on the two sides of these internal faults need be feared.

Earthquake Forces and Effects

The designers of the bridge reviewed the available data as to earthquake force and damage and formed the following opinions:

1. Material damage during earthquakes has been confined to structures of poor workmanship or to structures whose elements of greatest rigidity were not designed to transfer horizontal forces. Conversely, well-built structures, even if no definite allowance for seismic forces has been made in their design, have survived the most severe California earthquakes with no structural damage.

2. Japanese earthquakes appear to be of greater intensity than those in California; however, in Japan, structures designed for a horizontal acceleration of 10% of gravity have survived the heaviest earthquakes with no material structural damage. It appears safe to assume a smaller acceleration in designing against California earthquakes.

3. The maximum ground accelerations in Japan probably reach at least 50% and in California at least 25% of gravity.

4. It appears evident, therefore, that there must be some relieving factors not usually considered in a seismic design. These factors probably involve the mass and elasticity of the structure, its supports, and the period of the earth-

quake. It is a matter of observation that of two geometrically similar solids, the smaller is more likely to be overturned during an earthquake.

5. Although the ground motions during the violent phase of an earthquake are probably never truly harmonic, they may approach this condition for a limited number of vibrations.

6. The best information as to earthquake periods seems to be as follows:

(a) From the information now available, there is no conclusive evidence as to the existence of dominant ground periods, although certain periods may stand out more prominently than others.

(b) For the same character of ground, the shorter periods are damped out the more rapidly with increase in distance from the epicenter; hence, with increasing epicentral distances, the longer periods tend to prevail.

(c) In general, at equal epicentral distances, periods in alluvial soil are longer than in rock.

(d) At this particular location, considering the distance from the bridge to the San Andreas and Hayward faults and the fact that the piers are founded on rock or other solid strata, it appears probable that the period during the most violent phase of a strong earthquake would not exceed 0.7 sec.

(e) During the end phase of heavy shocks, almost pure simple harmonic motion, with a period of from 6 to 8 sec, may be expected at great distances from the epicenter. However, the information now available records no structural damage during such periods.

7. The evidence as to resonance and the dangers thereof is very conflicting. If resonance builds up to the extent assumed by some writers, the damage, during even a moderate earthquake, would be much greater than has been the experience. A possible explanation is that:

(a) A minor change in period such as may result from elastic yielding of foundations may be sufficient to destroy resonance.

(b) Close synchronism between the natural period of the structure and the period of forced vibration is required to establish resonance, especially in the higher modes.

8. In any vibratory motion, the maximum acceleration at any point exists only for an instant. In an elastic structure with a large percentage of its mass a considerable distance from the point of application of the earthquake motions, there will be a time lag before all parts of the structure are accelerated. Therefore, the maximum acceleration will not occur simultaneously in all parts of the structure.

9. Assuming resonant or near-resonant conditions, more or less damping is certain. There are no data sufficient to permit calculations of this effect.

DESIGN ASSUMPTIONS

Before proceeding with any calculations of stress, it was necessary to assume an hypothetical earthquake and the nature and intensity of the forces to which the structure would be subjected by such an earthquake and, also, in so far as possible, to analyze the resulting stresses. In many cases, the system is so complicated that a quantitative solution cannot be reached. For these cases, an attempt was made to assume forces higher than those that are believed probable.

Hypothetical Earthquake.—The earthquake forces were assumed as those resulting from a ground motion with a horizontal acceleration of 10% of gravity, a period of 1.5 sec, with a corresponding amplitude of 2.2 in. This is a longer period than is probable. It was chosen because, with other assumptions used in the design, it generally produced higher stresses than the same acceleration with a shorter period. However, the possibility of resonance at other periods was investigated.

Effect of Surrounding Water and Silt.—As the pier is moved by the rock on which it rests, the pier in turn displaces the surrounding water and, possibly, part of the surrounding silt.

For the analysis of these effects, studies were made of the paper by H. M. Westergaard, M. Am. Soc. C. E., on "Water Pressures on Dams During Earthquakes,"[3] and of the discussions of this paper. On the basis of the concept of "apparent mass," the equation in the discussion by Theodor von Kármán, M. Am. Soc. C. E., was used to determine the horizontal dimension of this mass. It was assumed that the boundary of the apparent mass was not affected below the mud line by the different specific weight encountered there. Equations for unit pressures due to earthquake were then developed, together with expressions for the shears and moments resulting therefrom. In the case of a bridge pier, the water has two paths of escape; vertically (which is the only possible one in the case of a dam), and around the ends. The dam analogy is very severe, since the actual movements would be a combination of the two paths. Because water is present on both sides of the pier, the acting forces were doubled; that is, "apparent masses" were assumed to exist simultaneously on opposite faces of the pier.

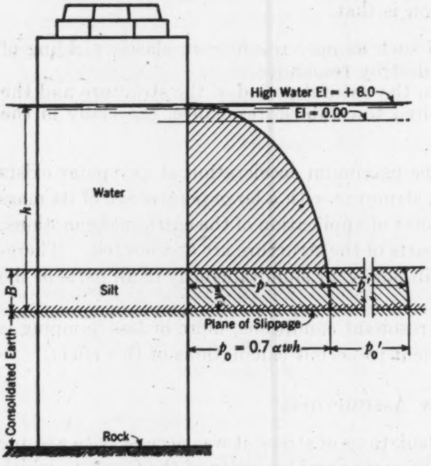


FIG. 3.—WATER AND SILT PRESSURES ACTING ON PIERS

In locations, such as tidal flats, there is evidence that during earthquakes the surface mud behaves as a viscous liquid and does not move as a unit with the underlying strata; that is, a slippage plane exists. The thickness of this moving mud is unknown, but considerations based on friction indicate that it is not great. The friction resulting from the weight of 60 ft of water above the mud may preclude the existence of a slippage plane. However, assuming such a plane, it is difficult to evaluate the force required to displace this mud. To be conservative, a slippage plane was assumed to exist at the top of the firm clays and sands, giving a moving mud layer in the general order of 20 ft.

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[3] *Transactions*, Am. Soc. C. E., Vol. 98 (1933), p. 418.

For purposes of stress analysis, it was assumed that this material acted as a liquid with a specific weight of 100 lb per cu ft and, further, that it might be acting in a phase 180° from the movement of the pier. Here again the forces were doubled, since mud is present on both sides of the pier. The forces acting on one face of a pier are indicated by Fig. 3. Let h = depth from water surface to plane of slippage; b = horizontal dimension of "apparent mass" at distance "y" above the plane of slippage. Then

$$b = 0.7\,(h^2 - y^2)^{0.5} \qquad (1)$$

Let p = unit pressure caused by earthquake, if the pier were surrounded to a depth h by water only; p' = additional unit pressure below the mud line; w = weight per unit volume of water; w' = weight per unit volume of silt; a = acceleration of the pier relative to that of the water (that is, the assumed earthquake acceleration); a' = acceleration of the pier relative to that of the mud or silt; $\alpha = \frac{a}{g}$; and $\alpha' = \frac{a'}{g}$.

The unit pressure on one face of the pier above the mud line becomes

$$p = \alpha\, w\, b = 0.7\,\alpha\, w\,(h^2 - y^2)^{0.5} \qquad (2)$$

Between the mud line and the plane of slippage, the unit pressure is

$$p + p' = \alpha'\, w'\, b \qquad (3)$$

From Eqs. 2 and 3,

$$p' = \alpha'\, w'\, b - \alpha\, w\, b \qquad (4)$$

$$\frac{p'}{p} = \frac{\alpha'\, w'}{\alpha\, w} - 1 = k \qquad (5)$$

and

$$p' = k\, p = 0.7\,\alpha\, w\, k\,(h^2 - y^2)^{0.5} \qquad (6)$$

At the plane of slippage, where $y = 0$,

$$p_0 = 0.7\,\alpha\, w\, h \qquad (7)$$

and

$$p_0' = 0.7\, k\,\alpha\, w\, h \qquad (8)$$

As stated, due to the presence of water on both faces of the pier, the total load per unit area was taken as equal to twice the pressures expressed by Eqs. 7 and 8. On this basis, the total shear resulting from the load p on two opposite faces of the pier is

$$V = 2\int_y^h p\, dy = 1.4\,\alpha\, w \int_y^h (h^2 - y^2)^{0.5}\, dy \qquad (9)$$

When $y = 0$, Eq. 9 becomes

$$V_0 = 1.1\,\alpha\, w\, h^2 \qquad (10)$$

Similarly, the shear resulting from p' is

$$V' = 2\int_y^B p'\, dy = 1.4\, k\,\alpha\, w \int_y^B (h^2 - y^2)^{0.5}\, dy \qquad (11)$$

in which B = depth of mud above the plane of slippage. When $y = 0$, Eq. 11 becomes

$$V_0' = 0.7\,k\,\alpha\,w \left[B\,(h^2 - B^2)^{0.5} + h \sin^{-1}\frac{B}{h} \right] \ldots\ldots\ldots (12)$$

The total shear at any elevation is

$$V_T = V + V' \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

The moment resulting from p is

$$M = 2 \int_y^h p\,y\,dy = 1.4\,\alpha\,w \int_y^h (h^2 - y^2)^{0.5}\,y\,dy \ldots\ldots\ldots (14)$$

When $y = 0$, Eq. 14 becomes

$$M_0 = 0.47\,\alpha\,w\,h^3 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

The moment resulting from p' is

$$M' = 2 \int_y^B p'\,y\,dy = 1.4\,k\,\alpha\,w \int_y^B (h^2 - y^2)^{0.5}\,y\,dy \ldots\ldots (16)$$

When $y = 0$, Eq. 16 becomes

$$M_0' = 0.47\,k\,\alpha\,w\,[h^3 - (h^2 - B^2)^{1.5}] \ldots\ldots\ldots\ldots (17)$$

The total moment is

$$M_T = M + M' \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (18)$$

Assuming no support from the consolidated earth (Fig. 3), the shear at any distance Z below the plane of slippage is

$$V_Z = V_0 + V_0' \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (19)$$

The moment at the same point is

$$M_Z = M_0 + M_0' + Z\,(V_0 + V_0') \ldots\ldots\ldots\ldots (20)$$

Earthquake Forces in the West Bay Superstructure

A general consideration of the action of the suspension bridge (Fig. 1(*a*)) is a necessary preface to the stress analysis. The San Francisco Anchorage is a massive concrete block and may be considered an upward extension of the rock. The rock mass of Yerba Buena Island forms the eastern anchorage, the cables being connected to grillages set in tunnels and securely held by the complete backfilling of the tunnels with concrete. The Central Anchorage is also massive but must be analyzed as a vertical cantilever 450 ft high. In the longitudinal direction, the towers are flexible columns and depend, to a large extent, on the cables acting as guys. In the transverse direction, they are comparatively stiff braced bents. The cables must be considered as transmitting heavy forces but as having small mass. Compared with their dimensions, the stiffening trusses are very flexible. At the towers and anchorages they are held in a transverse direction only. The floor system and the concrete floor slabs have expansion joints at 120-ft intervals. For small movements, the friction at these joints is probably such that the floor system participates in chord deformations. With heavy chord stresses and consequent larger deformations, the friction would be broken. In both horizontal and vertical directions, the trusses are partly fixed at the towers by the friction of the expansion joints.

An idea of the flexibility of the system may be gained by the movement under the loading conditions assumed for the design. The center of the longer spans has a calculated range of vertical motion of 25 ft; the top of Tower 2, a horizontal motion of 6.5 ft. With a wind of 90 miles per hr, the calculated horizontal deflections of the center spans are 9.5 ft. The stresses, especially in the suspended structure, are controlled by these distortions. Compared with the deflections previously given, the amplitudes of any earthquake motion are very small and suggest that the resulting earthquake stresses would be rather small as compared to the other forces.

Earthquake Forces Transmitted to Anchorages through Cables.—As the anchorages are accelerated in a longitudinal direction, they, in turn, accelerate the attached cables and cause accelerations in the various elements suspended from the cables. The problem is to find the additional forces set up in the cables by reason of these accelerations. Because of the complexity of the suspension structure and the different periods of each of its masses, it was idealized as shown in Fig. 4(*b*). The earthquake effect was obtained by increasing the cable pull F by the coefficient $\frac{a}{g} = \alpha = 0.10$ (see Fig. 4(*a*)).

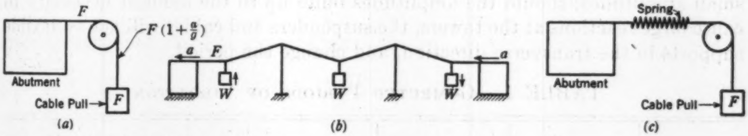

FIG. 4.—LONGITUDINAL CABLE PULL

As a second approach, the weight of the various spans may be replaced by equivalent weights at the centers of the three spans (Fig. 4(*b*)) and the increment of cable stress calculated when the anchorage has an acceleration of $\alpha\, g$, no allowance being made for the elongation of the cable. A calculation on this basis results in the change of stress in the cable of $0.7\, \alpha\, F$.

It is evident that the entire suspension system has many of the characteristics of a spring. The natural period (unloaded bridge) has been found by measurement to be 6.2 sec. The corresponding period for the fully loaded bridge would be 7.3 sec. The spring constant, or the additional cable stress for a 1-in. movement of the anchorage, is 35,000 lb for the two cables. Substituting in the spring equation, it is found that as far as dynamic action on the cables is concerned, the conditions may be replaced by the construction shown by Fig. 4(*c*), with the maximum earthquake force on the anchorage of $0.0265\, F$. It may be noted that, in any event, the different periods of the center and side span stiffening trusses would prevent resonance. It may be noted that increasing F by αF would have no material effect on either of the end anchorages. The center anchorage was designed on the basis (at high unit stresses) of one of the twin bridges being completely removed.

Earthquake Stresses in Towers.—In the transverse direction, the towers are subjected, during an earthquake, to stresses arising from the inertia of the suspended structure, of the cables, and of the towers themselves. The suspended spans are free to move longitudinally so that, except for the friction of the roadway expansion joints and of the suspended span bearings, there are no longitudinal forces at the floor level.

The natural periods of transverse vibration of the stiffening trusses were calculated for various assumptions of loading and end conditions as shown in Table 1. For pinned ends, the fundamental period is

$$T_1 = \frac{2\,L^2}{\pi}\left(\frac{w}{E\,I\,g}\right)^{0.5} \quad \dots\dots\dots\dots\dots\dots\dots\dots(21)$$

in which L = span length of stiffening truss, w = weight per foot of bridge, E = modulus of elasticity, I = moment of inertia of stiffening truss, and g = acceleration of gravity. For example, for assumption 2, $T_1 = 3.35 \times 10^6 \times 0.000\,000\,464 = 15.5$ sec. The first two harmonics are: $T_2 = \frac{T_1}{4} = 3.9$ sec; and, $T_3 = \frac{T_1}{9} = 1.7$ sec.

The periods in Table 1 are calculated on the basis that the suspenders are flexible and have no effect on the transverse vibrations. This is true only for small amplitudes; should the amplitudes build up to the amount necessary to cause large reactions at the towers, the suspenders and cables will act as flexible supports in the transverse direction, and change the period.

TABLE 1.—COMPUTED PERIODS OF VIBRATION

Assumption	Loading condition	End condition	Floor participation	PERIOD, IN SECONDS	
				Center span	Side spans
1	Loaded	Pinned	No	18.5	4.9
2	Unloaded	Pinned	No	15.5	4.1
3	Unloaded	Pinned	Yes	14.0	3.7
4	Unloaded	Fixed	No	6.8	1.8
5	Unloaded	Fixed	Yes	6.1	1.6

The periods of the center spans were measured under unloaded conditions and found to be approximately 9.0 sec. The corresponding amplitudes were very small, a maximum of 0.2 in. Probably, under these conditions, there would be no movements of the intermediate expansion joints, and the friction at the towers would be sufficient to approach the fixed-end condition. With amplitudes sufficient to cause considerable chord stresses, the movement of the floor expansion joints would reduce the floor participation, the friction at the towers would not be sufficient to maintain the fixed-end condition, and the periods would approach those of assumptions 1 or 2, Table 1, depending upon the loading on the bridge.

The end reactions for condition 1, Table 1, for various periods of forced vibration are shown by Fig. 5, which shows that the fundamental periods of both center and side spans are considerably greater than those at which earthquake damage may be anticipated. The values of the reaction used in the design and, also (for purpose of comparison), the wind reaction, are shown in Fig. 6.

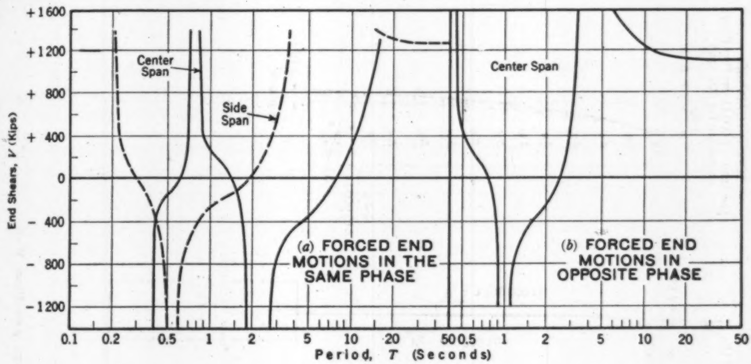

FIG. 5.—END SHEARS RESULTING FROM THE INERTIA OF A SUSPENDED STRUCTURE SUBJECTED TO FORCED VIBRATIONS AT ITS END SUPPORTS, WITH AMPLITUDES AND PERIODS CORRESPONDING TO 10% OF GRAVITY

The Appendix, under "Transverse Vibrations," shows the derivation of equations for moments and shears in the stiffening trusses. Completing the differentiation of Eq. 39*b* in the Appendix

$$V = \frac{d\,M}{E\,I} = E\,I\,\frac{d^3y}{dx^3} \qquad \text{(22)}$$

the equation for end shear is

$$V = E\,I\,m^3\,\frac{u_0}{2}\left[\left(\frac{1 - \cos m\,L}{\sin m\,L}\right) - \left(\frac{1 - \cosh m\,L}{\sinh m\,L}\right)\right] \qquad \text{(23)}$$

in which u_0 = amplitude of forced vibration. For example: When the forced period $T = 2.5$ sec, $L = 2{,}294.25$ ft; and $m = 3.41\,(10)^{-3}$; then, $m\,L = 7.823$ and $u_0 = 0.509$ ft for $\alpha = 0.10$. The end shear obtained by Eq. 23 would be 421,000 lb.

The cables are flexible. Until resonance is partly established it appears that, if a horizontal motion were imparted to them through the towers, the only inertia effect would be that imparted to a relatively small length of cable near the towers. Considering resonance and assuming small amplitudes, the theory of a tight string was applied, the actual tension in the cable and only the weight of the cable itself being considered. Under this assumption, the calculated natural fundamental periods for the bridge, loaded, are 5.9 and 3.1 sec for the center and side spans, respectively. It is also possible for the cable to swing as a compound pendulum; but the period, in the order of 15 sec for

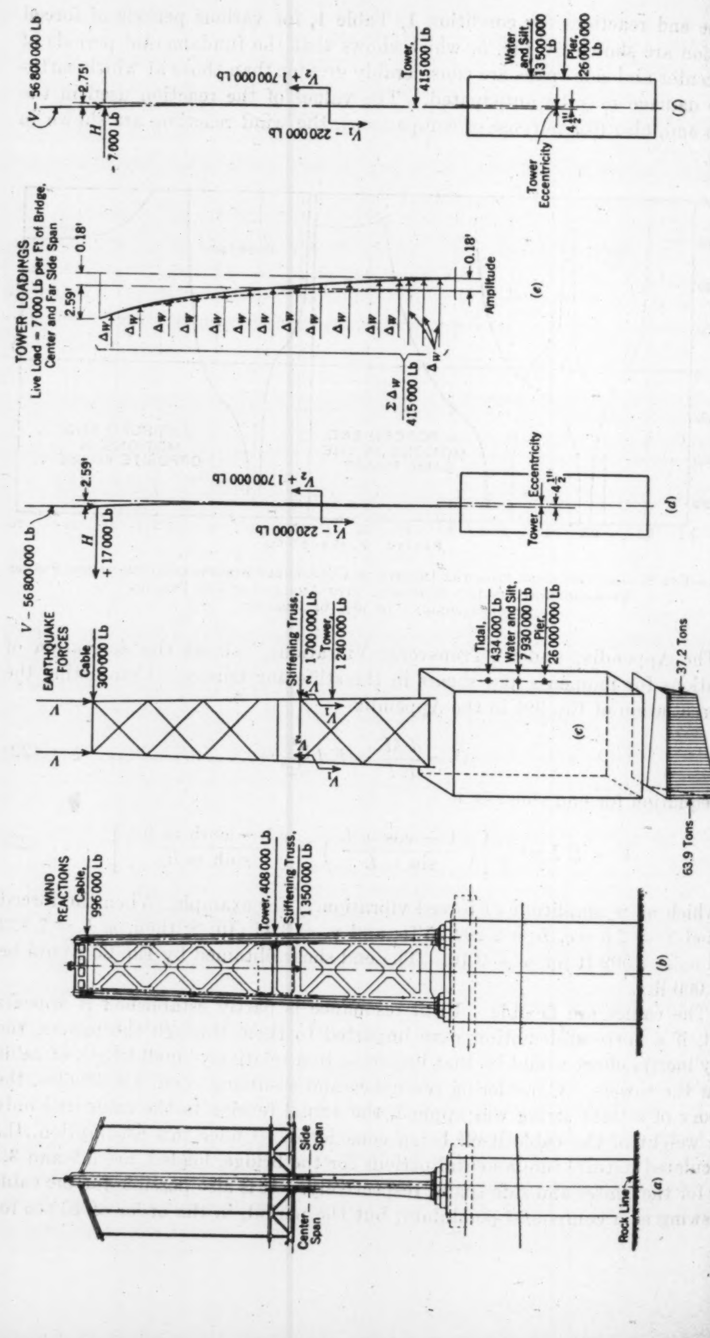
Center Span
Side Span
Rock Line
(a)
WIND REACTIONS
Cable, 996 000 Lb
Tower, 408 000 Lb
Stiffening Truss, 1 350 000 Lb
(b)
EARTHQUAKE FORCES
Cable 300 000 Lb
Stiffening Truss, 1 700 000 Lb
Tower, 1 240 000 Lb
Tidal, 434 000 Lb
Water and Silt, 7 930 000 Lb
Pier, 26 000 000 Lb
V
V_1
V_2
37.2 Tons
63.9 Tons
(c)
V - 56 800 000 Lb
H + 17 000 Lb
2.59'
V_2 + 1 700 000 Lb
V_1 - 220 000 Lb
Tower Eccentricity
4½"
(d)
TOWER LOADINGS
Live Load - 7 000 Lb per Ft of Bridge, Center and Far Side Span
0.18'
2.59'
Δw
ΣΔw 415 000 Lb
Amplitude
0.18'
(e)
V - 56 800 000 Lb
H - 7 000 Lb
2.75'
V_2 + 1 700 000 Lb
Tower, 415 000 Lb
V_1 - 220 000 Lb
Water and Silt, 13 500 000 Lb
Pier, 26 000 000 Lb
Tower Eccentricity
4½"

FIG. 6.—EARTHQUAKE FORCES ON TOWER AND PIER W-3

the center span, is entirely outside the range of damaging earthquake periods. Should the amplitudes reach any considerable amounts, the lateral resistance and mass of the suspended structure come into play, causing a change of periods.

To determine the properties of the main span cable considered as a compound pendulum, the following equations were applied: For center of gravity, assuming the cable curve to be a catenary, $y = \frac{f}{2}(e^{x/f} + e^{-x/f})$,

$$s\,h_g = \frac{f^2}{2}\sinh\frac{L}{f} + \frac{f\,L}{2} - 2\,f\,d\sinh\frac{L}{2\,f} \qquad (24)$$

in which s = length of cable between supports, and the other quantities are

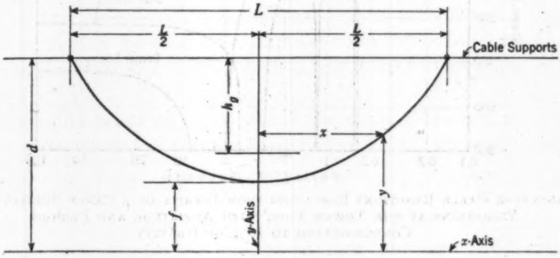

FIG. 7

as indicated by Fig. 7. For radius of gyration, r, about the axis through the cable supports,

$$s\,r^2 = \frac{f^3}{2}\left[\frac{1}{3}\sinh\frac{3\,L}{2\,f} + 3\sinh\frac{L}{2\,f}\right] - f^2\,d\sinh\frac{L}{f} - L\,f\,d + 2\,f\,d^2\sinh\frac{L}{2\,f} \qquad (25)$$

For this particular structure $h_g = 150.3$ ft and the period $T = 2\,\pi\left(\frac{C}{g}\right)^{0.5}$ = 15.0 sec in which $C = \frac{r^2}{h_g}$. This period checked the measured value as obtained by the seismometers.

Assuming resonance, the transverse reactions of the cables for forced vibrations of different periods are plotted on Fig. 8. From these curves, a horizontal reaction of 300 kips was chosen. Equations for the computation of these forces are shown in the Appendix under the heading "Flexible Vibrating String."

To the stresses in the tower resulting from the suspended span and cable reactions, those from transverse forces equal to 10% of the weight of the tower were added, giving the total tower stresses from an earthquake in the transverse direction.

In a longitudinal direction, except for a negligible amount of friction at the roadway level, the only additional forces acting on the tower during an earthquake are those arising from the inertia of the tower itself and such changes as there may be in the unbalanced horizontal component of the cables. Fig. 6(f) shows the deflected position of the tower under the most severe assumption of live load and temperature. The tower tops are held in position by the cables acting as guys. Compared to the other forces, the force required to deflect the towers is very small.

In a longitudinal earthquake, one may assume that the base of the tower is moved and that the top of the tower remains practically fixed in space. The earthquake forces on the various elements of the tower (see Fig. 6(*e*)) are assumed as proportional to their respective amplitudes.

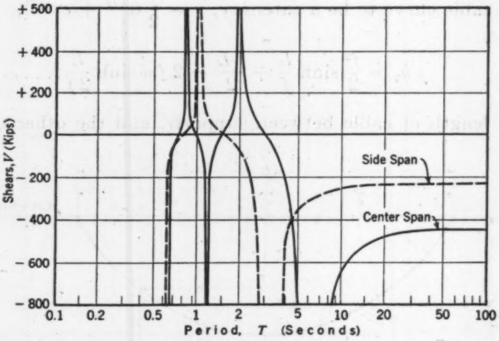

FIG. 8.—TRANSVERSE CABLE REACTIONS RESULTING FROM INERTIA OF A CABLE SUBJECTED TO FORCED VIBRATIONS AT THE TOWER TOPS, WITH AMPLITUDE AND PERIODS CORRESPONDING TO 10% OF GRAVITY

The value of the displacement Δ and all forces except H can be determined by the "Modified Dana Method"[4] and H can be found by the following equations: For a horizontal force at the end of a cantilever, the equation for the deflection is:

$$\Delta_H = \sum_0^L \frac{M\,y\,dy}{E\,I} = \sum_0^L \frac{H\,y^2\,dy}{E\,I} \quad \text{.............} (26)$$

for a vertical force:

$$\Delta_V = \sum_0^L \frac{M\,y\,dy}{E\,I} = \sum_0^L \frac{V\,(\Delta - \delta_y)\,y\,dy}{E\,I} \quad \text{.............} (27)$$

and, for a moment:

$$\Delta_M = \sum_e^L \frac{M\,dy}{E\,I} = \sum_e^L \frac{M\,(y - e)\,dy}{E\,I} \quad \text{.............} (28)$$

In Eqs. 26, 27, and 28: Δ = horizontal displacement of top of tower; y = vertical distance from top of tower; δ_y = horizontal displacement at distance y from top of tower; V = vertical load; M = moment; and e = vertical distance from top of tower to a point where moment load is applied. From these three fundamental equations the various forces due to wind, loading, earthquake, etc., were obtained. The displacements for this particular structure are shown in Fig. 6. Each value of Δ_W in Fig. 6(*e*) was assumed proportional to its respective amplitude.

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[4] "Tests on Structural Models of Proposed San Francisco-Oakland Suspension Bridge," by George E. Beggs, Raymond E. Davis, M. Am. Soc. C. E., and Harmer E. Davis, Assoc. M. Am. Soc. C. E., *Publications in Engineering*, Univ. of California, Vol. 3, p. 151.

East Bay Crossing

The principal structures in the East Bay Crossing consist of the cantilever structure and the five 508-ft spans which have a common junction at Pier E-4, each system being anchored in a longitudinal direction at Pier E-1 and Pier E-9, respectively. Consideration may be given to the conditions before resonance is established, stresses during resonance, and the factors inherent in the construction to destroy resonance.

The natural longitudinal period of the unloaded cantilever structure has been determined by measurement as approximately 1.5 sec. With the bridge fully loaded, this period would be increased to about 1.8 sec. This has been interpreted as indicating that if an impulse were applied at Pier E-1, it would travel across the span and reach Pier E-4 in 0.9 sec. Therefore, with a period 1.8 sec, and with a condition of no acceleration at either end, the maximum acceleration would occur at the center of the span. The assumption has been made, therefore, that for the west anchor arm, an average acceleration of $\frac{2}{\pi}$ times 10% could be used instead of 10% on the entire structure. The resulting forces are assumed to be statically applied. By this assumption, the maximum horizontal force applied to Pier E-1 from the inertia forces of the cantilever structure, including live load, is 5,000,000 lb. The similar assumptions for the longitudinal stresses in the 508-ft spans are shown in Fig. 6(*d*).

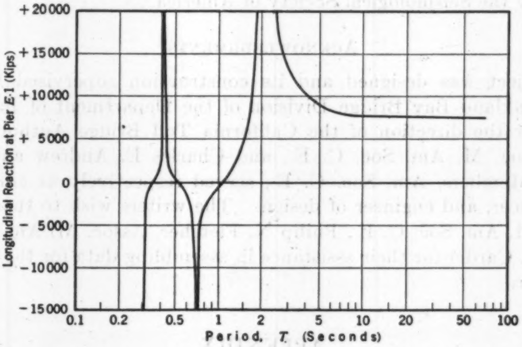

FIG. 9.—REACTIONS AT PIER E-1 RESULTING FROM THE INERTIA OF THE CANTILEVER STRUCTURE SUBJECTED TO FORCED LONGITUDINAL VIBRATIONS AT THAT PIER, WITH AMPLITUDES AND PERIODS CORRESPONDING TO 10% OF GRAVITY

The horizontal forces from the cantilever, acting on Pier E-1 in the case of continued forced harmonic vibrations with no allowance for damping, are shown in Fig. 9. Equations for the calculation of these forces are shown in the Appendix, under the heading "Longitudinal Vibrations."

In the transverse direction, the design was made on the basis of a horizontal static force of 10% gravity. With the elasticity of the structures, especially the cantilever, some reduction could have been safely made. The measured natural period of this structure (unloaded) is 3.44 sec. Under load, it would be increased to about 3.9 sec.

The following are among the reasons that make resonance improbable:

(1) The natural periods of the structure (fundamental mode) are considerably in excess of those of the probable ground motions; and

(2) The floor expansion joints will have an action and effect similar to the foregoing in the case of the suspension structure.

Effect of Earthquake Stresses on Design.—Although the earthquake stresses in the various parts of the bridge when calculated in accordance with the foregoing assumptions are of considerable magnitude, they had a comparatively small effect on the proportions of the bridge. Generally, the maximum foundation pressures occur when seismic forces are considered. On the other hand, the piers are no larger than would ordinarily be used considering the size of the superstructure. The ratio of the exposed area to the mass of the superstructure is such that the horizontal forces from wind are of the same order as those from the earthquake assumptions. For these reasons, the increased cost arising from provision for a 10% seismic coefficient was very small, probably less than 5%.

Vibration Periods of Bridge.—In the foregoing, reference has been made to the measured vibration periods of the bridge. These measurements were made by engineers of the U. S. Coast and Geodetic Survey and the results published by the Seismological Society of America.[5]

ACKNOWLEDGMENTS

The project was designed and its construction supervised by the San Francisco-Oakland Bay Bridge Division of the Department of Public Works acting under the direction of the California Toll Bridge Authority. C. H. Purcell, Assoc. M. Am. Soc. C. E., and Charles E. Andrew and Glenn B. Woodruff, Members, Am. Soc. C. E., served respectively as chief engineer, bridge engineer, and engineer of design. The writers wish to thank Franklin P. Ulrich, M. Am. Soc. C. E., Philip N. Fletcher, Assoc. M. Am. Soc. C. E., and Dean S. Carder for their assistance in assembling data for the preparation of this paper.

APPENDIX

THEORY

The derivations of some of the formulas that have been used in calculating the effects of forced harmonic vibrations on certain parts of the bridge are given herein. In these derivations, ideal cases only have been considered; that is, the bridge structure has been assumed to be of such dimensions that it can be considered as having the characteristics of a thin rod, and the cable (for transverse vibrations) has been assumed as a flexible vibrating string. Transverse distortions in longitudinal vibrations, variations in areas and moments of inertia, some of the effects of the addition of inert masses to the bars, and similar other factors that will affect numerical values, have not been considered. The effect of damping has been omitted entirely. For these reasons, the numerical results obtained for any particular period of forced vibration may be considerably in error. However, if calculations are made for a series of different periods, the general characteristics of the induced stresses thus determined can be used as one of the guides in the selection of design stresses.

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[5] "Observed Vibration of Bridges," by Dean S. Carder, *Bulletin of the Seismological Society of America*, Vol. 27, No. 4, October, 1937.

TRANSVERSE VIBRATIONS

The differential equation for the transverse vibrations of thin uniform rods is:

$$\frac{\partial^4 y}{\partial x^4} + \frac{w_r}{E\,I\,g}\frac{\partial^2 y}{\partial t^2} = 0 \qquad (29)$$

in which: w_r = weight per unit length of rod; E = modulus of elasticity; I = moment of inertia of the rod; and t = time. To solve this equation, assume $y = u \cos p\,t$ in which amplitude u is a function of x only. Then

$$\frac{\partial^2 y}{\partial t^2} = -\,u\,p^2 \cos p\,t \qquad (30a)$$

and

$$\frac{\partial^4 y}{\partial x^4} = \frac{\partial^4 u}{\partial x^4}\cos p\,t \qquad (30b)$$

Substituting these values in Eq. 29,

$$\frac{d^4 u}{dx^4} - \frac{p^2\,w_r\,u}{E\,I\,g} = 0 \qquad (31a)$$

Let $\frac{p^2\,w_r}{E\,I\,g} = m^4$; then

$$\frac{d^4 u}{dx^4} - m^4\,u = 0 \qquad (31b)$$

A solution of Eq. 31*b* is:

$$u = A\cos m\,x + B\sin m\,x + H\cosh m\,x + K\sinh m\,x \qquad (32)$$

Natural Period of Vibration of a Rod Pinned at Both Ends.—The end conditions are: At $x = 0$ and at $x = L$, $y = 0$ (no deflection), and $\frac{\partial^2 y}{\partial x^2} = 0$ (moment = 0). Therefore, $u = 0$ and $\frac{d^2 u}{dx^2} = 0$, when $x = 0$ and $x = L$.

Applying the foregoing conditions at $x = 0$ gives $A = H = 0$. Applying the same conditions at $x = L$ gives:

$$0 = A \cos m\,L + B \sin m\,L + H \cosh m\,L + K \sinh m\,L \ldots \text{(33a)}$$

and

$$0 = -A \cos m\,L - B \sin m\,L + H \cosh m\,L + K \sinh m\,L \ldots \text{(33b)}$$

Since $A = H = 0$, the addition and subtraction of these two equations give, respectively,

$$2\,K \sinh m\,L = 0 \ldots\ldots \text{(34a)}$$

and

$$2\,B \sin m\,L = 0 \ldots\ldots \text{(34b)}$$

Since $\sinh m\,L$ cannot be zero, K must be zero. If $B = 0$, y is always zero and the rod is at rest. Therefore $\sin m\,L$ must be zero. This gives: $m\,L = \pi$ or $2\,\pi$ or $3\,\pi$ etc. $\cdots$ or $n\,\pi$; $m = \frac{\pi}{L}$; and $m^4 = \frac{\pi^4}{L^4} = \frac{p^2\,w_r}{E\,I\,g}$. Therefore, for the fundamental mode of vibration,

$$p = \frac{\pi^2}{L^2 \left(\frac{w_r}{E\,I\,g}\right)^{0.5}} \ldots\ldots \text{(35)}$$

For higher modes when $m = \frac{n\,\pi}{L}$,

$$p_n = \frac{n^2\,\pi^2}{L^2 \left(\frac{w_r}{E\,I \cdot g}\right)^{0.5}} \ldots\ldots \text{(36)}$$

The natural period of vibration is

$$T = \frac{2\,\pi}{p_n} = \frac{2\,L^2}{n^2\,\pi} \left(\frac{w_r}{E\,I\,g}\right)^{0.5} \ldots\ldots \text{(37)}$$

Rod Subjected to Simple Harmonic Motion in a Transverse Direction at Each End.—Assume harmonic motions of both ends to be in phase. The end conditions are: At $x = 0$ and at $x = L$, $y = u_0 \cos p\,t$, and $\frac{\partial^2 y}{\partial x^2} = 0$. Therefore, $u = u_0$ and $\frac{d^2 u}{dx^2} = 0$, when $x = 0$ and $x = L$.

From these end conditions it is found that

$$A = H = \frac{u_0}{2} \ldots\ldots \text{(38a)}$$

$$K = \frac{u_0}{2} \frac{(1 - \cosh m\,L)}{\sin m\,L} \ldots\ldots \text{(38b)}$$

and

$$B = \frac{u_0}{2} \frac{(1 - \cos m\,L)}{\sin m\,L} \ldots\ldots \text{(38c)}$$

When the constants in Eqs. 38 are known, moments and shears can be calculated by means of the following relationships:

$$M = E\,I\,\frac{d^2y}{dx^2} \qquad (39a)$$

and

$$V = \frac{dM}{dx} = E\,I\,\frac{d^3y}{dx^3} \qquad (39b)$$

Rod Subjected to Simple Harmonic Motion in a Transverse Direction at Each End.—Assume harmonic motions at the ends to be 180° out of phase with each other. Then the end conditions will be (when $x = 0$, $y = u_0 \cos p\,t$, or $u = u_0$) $\frac{\partial^2 y}{\partial x^2} = 0$, or $\frac{d^2u}{dx^2} = 0$. When $x = L$, $y = u_0 \cos (p\,t + \pi)$ or $u = -u_0$, and $\frac{\partial^2 y}{\partial x^2} = 0$, or $\frac{d^2u}{dx^2} = 0$.

From these end conditions it is found that:

$$A = H = \frac{u_0}{2} \qquad (40a)$$

$$K = -\frac{u_0}{2}\left(\frac{1 + \cosh m\,L}{\sinh m\,L}\right) \qquad (40b)$$

and

$$B = -\frac{u_0}{2}\left(\frac{1 + \cos m\,L}{\sin m\,L}\right) \qquad (40c)$$

LONGITUDINAL VIBRATIONS

Let u = longitudinal displacement at any point along the rod; x = distance along the rod; $\frac{w_r}{g}$ = mass of rod per unit volume; and E = modulus of elasticity. Then $E\,\frac{\partial^2 u}{\partial x^2}\,dx = \frac{w_r}{g}\,dx\,\frac{\partial^2 u}{\partial t^2}$; or

$$\frac{\partial^2 u}{\partial t^2} = c^2\,\frac{\partial^2 u}{\partial x^2} \qquad (41)$$

in which

$$c^2 = \frac{g\,E}{w_r} \qquad (42)$$

Assume $u = \xi \sin p\,t$ where ξ is a function of x only and p is a constant. Then

$$\frac{\partial^2 u}{\partial t^2} = -\,p^2\,\xi \sin p\,t \qquad (43a)$$

and

$$\frac{\partial^2 u}{\partial x^2} = \frac{d^2\xi}{dx^2} \sin p\,t \qquad (43b)$$

Therefore, $-p^2\,\xi \sin p\,t = c^2 \dfrac{d^2\xi}{dx^2} \sin p\,t$; or

$$\frac{d^2\xi}{dx^2} = -\frac{p^2}{c^2}\,\xi \tag{44}$$

and

$$\xi = C_1\,e^{q} + C_2\,e^{-q} \tag{45a}$$

$$\xi = C_1 \cos\frac{p\,x}{c} + C_1\,i \sin\frac{p\,x}{c} + C_2 \cos\frac{p\,x}{c} - C_2\,i \sin\frac{p\,x}{c} \tag{45b}$$

$$\xi = A \cos\frac{p\,x}{c} + B \sin\frac{p\,x}{c} \tag{45c}$$

in which, to simplify typography, $q = \dfrac{i\,p\,x}{c}$.

Natural Periods of Vibration for Rod Fixed at One End and Free at the Other.—The end conditions are: Where $x = 0$, $u = 0 = \xi$, and where $x = L$, $\dfrac{\partial u}{\partial x} = 0 = \dfrac{d\xi}{dx}$. Furthermore, $0 = A$ (from $x = 0$ and $u = 0$) $= \dfrac{B\,p}{c} \cos\dfrac{p\,L}{c}$ $\left(\text{from } x = L \text{ and } \dfrac{\partial u}{\partial x} = 0\right)$.

If $B = 0$, there is no motion; therefore $\cos\dfrac{p\,L}{c} = 0$ and $\dfrac{p\,L}{c} = \dfrac{\pi}{2}$, or $\dfrac{3\,\pi}{2}$, or $\dfrac{5\,\pi}{2}$, etc. $= \dfrac{(2\,n - 1)}{2}\,\pi$, in which n is an integer. Also,

$$p = \frac{(2\,n - 1)\,\pi\,c}{2\,L} \tag{46}$$

The natural period $T_n = \dfrac{2\,\pi}{p}$; and, for the fundamental period,

$$T_1 = 2\,\pi \times \frac{2\,L}{\pi\,c} = \frac{4\,L}{c} \tag{47}$$

Rod Subjected to Harmonic Motion in a Longitudinal Direction at One End (Other End Free).—Let u_0 = half amplitude of forced harmonic motion at $x = 0$ (that is, u_0 is the maximum displacement). The end conditions are: When $x = 0$, $\xi = u_0$ and $A = u_0$. At the free end ($x = L$), there is no stress. Therefore, $\dfrac{d\xi}{dx_{(x=L)}} = 0$, so that

$$\frac{d\xi}{dx} = -\frac{A\,p}{c} \sin\frac{p\,x}{c} + \frac{B\,p}{c} \cos\frac{p\,x}{c} \tag{48a}$$

$$0 = \frac{-\,u_0\,p}{c} \sin\frac{p\,L}{c} + \frac{B\,p}{c} \cos\frac{p\,L}{c} \tag{48b}$$

and

$$B = \frac{u_0 \sin\dfrac{p\,L}{c}}{\cos\dfrac{p\,L}{c}} = u_0 \tan\frac{p\,L}{c} \tag{48c}$$

Then

$$u = u_0 \left(\cos \frac{p\,x}{c} + \tan \frac{p\,L}{c} \sin \frac{p\,x}{c} \right) \sin p\,t \ldots\ldots\ldots\ldots (49a)$$

and

$$\frac{\partial u}{\partial x} = \frac{u_0\,p}{c} \left(-\sin \frac{p\,x}{c} + \tan \frac{p\,L}{c} \cos \frac{p\,x}{c} \right) \sin p\,t \ldots\ldots\ldots (49b)$$

The total longitudinal stress $= A\,E\,\frac{\partial u}{\partial x}$, in which A = area. At $x = 0$, the maximum longitudinal stress equals

$$\frac{A\,E\,u_0\,p}{c} \tan \frac{p\,L}{c}\,.$$

Flexible Vibrating String

Fig. 10 shows an elementary length of a vibrating string. Let T = tension in string; L = length (distance between supports); and ρ = mass per unit length of string. The forces acting on length ds in the y-direction are: $\rho\,ds\,\frac{\partial^2 y}{\partial t^2}$ due to acceleration, and $T\,\frac{\partial^2 y}{\partial x^2}\,dx$ due to change in direction of T in the distance dx. Since the angles are small, ds can be assumed to be equal to dx and $\rho\,dx\,\frac{\partial^2 y}{\partial t^2}$ can be substituted for $\rho\,ds\,\frac{\partial^2 y}{\partial t^2}$. Then

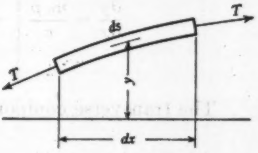

FIG. 10

$$\rho\,dx\,\frac{\partial^2 y}{\partial t^2} = T\,\frac{\partial^2 y}{\partial x^2}\,dx \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (50)$$

or,

$$\frac{\partial^2 y}{\partial t^2} = c^2\,\frac{\partial^2 y}{\partial x^2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (51)$$

in which $c^2 = \frac{T}{\rho}$. A solution of Eq. 51 is

$$y = \left(A \cos \frac{p\,x}{c} + B \sin \frac{p\,x}{c} \right) \sin p\,t \ldots\ldots\ldots\ldots\ldots (52)$$

Natural Periods of Vibration of a String Under Tension.—The end conditions are: When $x = 0$, $y = 0$; therefore, $A = 0$. When $x = L$, $y = 0$; therefore, $B \sin \frac{p\,L}{c} = 0$.

If $B = 0$, there is no motion; therefore, $\sin \frac{p\,L}{c}$ must equal zero, and $\frac{p\,L}{c} = \pi$ or $2\,\pi$ or $3\,\pi$, etc.; and $p = \frac{n\,\pi\,c}{L}$ (in which n is an integer); natural period $= \frac{2\,\pi}{p} = \frac{2\,L}{n\,c}$; and fundamental period $T_1 = \frac{2\,L}{c}$.

Flexible String Under Tension and Subjected to Simple Harmonic Motion in a Transverse Direction at Each End.—Let u_0 = the half amplitude of forced harmonic motion at each end of a string. The end conditions are: When $x = 0$, $y = u_0 \sin p\,t$; therefore, $A = u_0$. When $x = L$, $y = u_0 \sin p\,t$; and $u_0 = u_0 \cos \frac{p\,L}{c} + B \sin \frac{p\,L}{c}$. Therefore,

$$B = \frac{u_0\left(1 - \cos\frac{p\,L}{c}\right)}{\sin\frac{p\,L}{c}} \qquad (53)$$

Then

$$y = u_0 \left[\cos\frac{p\,x}{c} + \frac{1 - \cos\frac{p\,L}{c}}{\sin\frac{p\,L}{c}} \sin\frac{p\,x}{c}\right] \sin p\,t \qquad (54a)$$

and

$$\frac{\partial y}{\partial x} = \frac{u_0\,p}{c}\left[-\sin\frac{p\,x}{c} + \frac{1 - \cos\frac{p\,L}{c}}{\sin\frac{p\,L}{c}} \cos\frac{p\,x}{c}\right] \sin p\,t \qquad (54b)$$

The transverse component of tension $= T\,\frac{\partial y}{\partial x}$; when $x = 0$,

$$\frac{\partial y}{\partial x} = \frac{u_0\,p}{c}\,\frac{\left(1 - \cos\frac{p\,L}{c}\right)}{\sin\frac{p\,L}{c}};$$

and, when $x = L$,

$$\frac{\partial y}{\partial x} = \frac{-\,u_0\,p}{c}\,\frac{\left(1 - \cos\frac{p\,L}{c}\right)}{\sin\frac{p\,L}{c}}.$$

DISCUSSION

Maurice A. Biot,[6] Esq. (by letter).—It was found that for a simple structure with one degree of freedom, subjected to a horizontal motion, the maximum shear force due to an earthquake could be expressed as

$$V = m\,A(T) \qquad (55)$$

in which m is the total mass, and A an acceleration function of the period of the structure but characteristic of a given earthquake. This quantity A is referred to as the "equivalent acceleration" for a simple resonator because of the fact that, for a structure of given period T_1, a static acceleration of value $A(T_1)$ produces the same stress as the earthquake on that particular structure. Eq. 55 may be written in a different form as

$$V = V_g\,\frac{A(T)}{g} \qquad (56)$$

in which V_g represents the maximum shear under a static horizontal force due to gravity.

The equivalent acceleration $A(T)$, plotted as a function of the period, is a characteristic curve of the earthquake called a spectrum. Such spectrum curves were evaluated by the use of a mechanical analyzer for the earthquakes of Helena, Mont., on October 31, 1935, and Ferndale, Calif., on September 11, 1938, for which the records show an acceleration peak of 0.16 g and 0.17 g, respectively. It was found that, for periods greater than 0.2 sec, the envelop of both spectra could be approximately represented by the formula

$$A = \frac{0.2\,g}{T} \qquad (57)$$

in which T is the period in seconds. Eq. 56 may be generalized so as to take into consideration structures with more than one degree of freedom. The maximum shear and the bending moments in each mode of vibration excited by the earthquake may be written, respectively,

$$V_n = C_n\,V_g\,\frac{A}{g}\,(T_n) \qquad (58a)$$

and

$$M_n = B_n\,M_g\,\frac{A}{g}\,(T_n) \qquad (58b)$$

in which V_g and M_g are the maximum shear and bending moments produced by a static horizontal force equal to gravity, and T_n is the period of the particular mode considered. The coefficients C_n and B_n may be called "efficiency factors," depending on the type of structure and the order of the excited mode. Values of these coefficients for a cable and a pin-ended uniform truss are given in Table 2. Using the data in the paper, one can apply these results to evaluate an upper limit for the stresses which would be produced by an earthquake of the Helena or Ferndale type in the San Francisco-Oakland Bay Bridge.

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[6] California Inst. of Technology, Pasadena, Calif. On leave from Columbia Univ., New York, N. Y.

Eqs. 55 to 58 apply only where the ground moves rigidly and would not cover the case in which the towers of the bridge and anchorages are moving out of phase. However, in the latter case it may be verified that the stresses are generally less than if the ground were rigid.

TABLE 2.—COEFFICIENT FOR A CABLE AND A PIN-ENDED UNIFORM TRUSS

Order of excited mode	$n = 1$	$n = 3$	$n = 5$
C_n for cable....................	0.816	0.0905	0.0326
C_n for truss....................	0.816	0.010	0.0013
B_n for truss....................	1.03	0.012	0.0016

Center-Span Truss.—The measured period in the center-span truss being 9.0 sec and the coefficient $C_1 = 0.816$ (from Eqs. 57 and 58*a*), the maximum shear in the fundamental mode is $V_1 = 0.017\ V_g$. This is the same as that produced by a static force of 1.7% of gravity.

Similarly for the bending moment—applying Eq. 58*b* with $B_1 = 1.03$—$M_1 = 0.022\ M_g$, it is possible to state that the equivalent accelerations for this case are 1.7% and 2.2% of gravity.

Side-Span Truss.—According to Table 1, a probable period of the side span is 3 sec. This corresponds to end conditions intermediate between pin-ended and fixed. The maximum shear and bending moment for the fundamental mode are $V_1 = 0.053\ V_g$, and $M_1 = 0.067\ M_g$. The equivalent accelerations for the shear force and bending moments are, respectively, 5.3% and 6.7% of gravity.

Cables.—The periods for the center-span and side-span cables are, respectively, 5.9 and 3.1 sec. Hence, the corresponding maximum shear stresses are the fundamental mode of these cables: $V_1 = 0.027\ V_g$, and $V_1 = 0.053\ V_g$. The equivalent accelerations are 2.7% and 5.3% of gravity.

Higher Modes.—The stresses in the higher modes are smaller than in the fundamental. Consider, for instance, the case of the side-span truss. The excited symmetric mode next to the fundamental will have a period of about 0.3 sec. Using the coefficients $C_3 = 0.010$ and $B_3 = 0.012$ of Table 2, the shear and the bending moment are found to be: $V_3 = 0.0066\ V_g$, and $M_3 = 0.008\ M_g$. These stresses are negligible compared to those in the fundamental mode. A similar conclusion holds for the cables.

Conclusions.—The writer has computed the stresses that the Helena and Ferndale earthquakes would produce in the San Francisco-Oakland Bay Bridge and has found that a stress corresponding to a static force of 6.7% of gravity could be produced in the side-span truss. The peak acceleration of the earthquakes considered is about 17% of gravity. Since stronger earthquakes with a peak intensity of 30% to 40% of gravity are not improbable, it seems that one should consider side-span stresses corresponding to an equivalent acceleration of about 10% to 12% of gravity. It must be remembered, however, that the effect of the damping has been neglected. This effect for large stresses can be quite considerable, due to the friction at the expansion joints, local plastic deformations, and the dissipation of energy by radiation in the ground through the foundation and the anchorage. Further research is necessary before the effect of the damping on earthquake stresses, and various other reducing factors, can be evaluated correctly.

FRANKLIN P. ULRICH,[7] M. AM. SOC. C. E. (by letter).—Although past history shows that earthquake damage has been chiefly confined to structures of poor construction or poor design, there is no assurance that what engineers now consider good construction is immune from future earthquake damage.

There are several factors to consider in possible earthquake damage, including intensity of earthquake, nearness to center of disturbance, and condition of the structure, especially foundation conditions.

In an attempt to learn more about some of these factors, 62 strong-motion seismographs have been set up in Western United States. These instruments will operate during the strongest earthquakes, giving information vital to structural design, such as acceleration, amplitude of motion, period of oscillation and duration of shock and its various parts. Although these instruments have been in operation only a few years, some surprising information has been obtained. In the El Centro (Calif.) shock in 1940 these instruments recorded a resultant horizontal acceleration of 38% of gravity. Some schools in this district that were designed to withstand continuous horizontal accelerations of 10%, and other buildings where the earthquake factor was not especially considered in the design, came through the shock with no serious structural damage. There is a factor in every structure which might be called the "inherent strength," and which is capable of resisting earthquake stress, effectively, to some unknown degree. Hence, a structure that is designed to resist an earthquake acceleration of 10% may, and no doubt would, withstand an earthquake acceleration several times that amount, providing it accompanied relatively short-period motion, as it usually does.

The second factor, mentioned previously, is the nearness to the epicenter. It is quite certain that all normal structures set directly on an earthquake fault, such as the San Andreas, would be seriously damaged by an earthquake of the intensity of that in 1906. It would seem equally certain that a relative lateral motion of two adjacent piers equal to that measured in the 1906 earthquake would seriously damage the San Francisco-Oakland Bay Bridge.

The Fault Map of the Seismological Society of America does show a "probable active fault" along the west side of Yerba Buena Island, and in recent years there has been at least one minor shock along this fault, near the Oakland Mole. However, in this case major active faults are situated relatively close on either end of the bridge, and it would appear to be reasonable logic to expect that large earth movements would occur along those faults and not on a minor fault between these major faults. Minor earthquakes are frequent in this region, but are generally confined to the major fault zones.

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[7] Chf., Seismological Field Survey, U. S. Coast and Geodetic Survey, San Francisco, Calif.

Information on the seismicity of this region and other regions is being obtained for the engineer and designer through the network of sensitive seismograph stations and through the extensive program of reports of felt earthquake shocks.

The third factor of foundation conditions and condition of structure is perhaps not as vital in this bridge as on normal buildings where uneven settlement would impose excessive stresses in some of the structural members, or structural deterioration would weaken the structure to a danger point—especially at the time of a strong earthquake. In this bridge the foundations of the west piers extend to rock and the foundations of the east piers extend to rock or comparatively solid strata so that there should be no settlement problem. The durability of concrete under salt water for a long period of time might be an uncertain factor, but tests made on the concrete used in these bridge piers showed a very satisfactory resistance to this deterioration.

Summary.—(1) In the light of past experience and the present knowledge of destructive ground motion the assumed earthquake factor of 10% at the period of 1.5 sec would probably be sufficient to resist an earthquake as great as would be expected in this region; (2) with major active faults at some distance from each end of the bridge it would be logical to believe that the bridge will not be subjected to uneven movements of adjoining parts of the structure; (3) with the piers extending to bedrock, or solid strata, there should be no settling of the bridge, especially uneven settlement; and (4) the weakening of the structure through deterioration of the concrete is an uncertain factor, but has been carefully checked in preconstruction tests.

LEON S. MOISSEIFF,[8] M. AM. SOC. C. E. (by letter).—The scientific designing of structures is based on the analytical and experimental knowledge of the behavior of a given construction built of dependable materials of known strength characteristics and attached to a foundation of ascertained bearing resistance when subjected to the action of well-defined forces. Analyzing the foregoing statement one finds that it implies a number of major assumptions in knowledge. When dealing with the resistance of structures to earthquakes, the assumption made by the designer of the magnitude and kind of forces caused by earthquakes which may be encountered in the locality, and reasonably must be provided for in the structure, is of deciding importance. Engineers generally realize that the data on earthquakes are as yet meager and the knowledge of their effects is still vague. There are indications, however, that more extensive and well-planned observations which are being furthered at the present time, as well as deepened study, will tend to enlarge and precise the scope of information and furnish more assurance of the correctness of the assumptions made.

In the meantime, engineers who build structures in earthquake regions (and almost all regions are more or less vulnerable) are compelled to make the most of the available data on earthquakes. By making further simplifying assumptions as to the kind of action of the quake forces, their maximum effect is transformed mathematically into an equivalent horizontal force, the intensity of which is a fraction of the acceleration of gravity. For the location of the San Francisco-Oakland Bay Bridge this force was assumed to be 10% of gravity. In structural terms this means that a rigid body, which does not deform elastically, would be subjected to a horizontal force of 10% of its weight acting at the center of gravity.

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[8] Cons. Engr., New York, N. Y.; and Member of Board of Cons. Engrs., San Francisco-Oakland Bay Bridge.

Attention is called to the fact that, in the present state of knowledge, the effect produced on the structure by an earthquake cannot as yet be considered as a "well-defined" force. Designers are compelled to content themselves with an assumed statical force. Having made this assumption, it becomes prudent to be somewhat lavish as to the intensity of the force, the more so when one finds that the provision for it in the structure causes relatively small expense.

What appears within the present knowledge of earthquake effects to be an ample intensity has been adopted in the design for the San Francisco-Oakland Bay Bridge. The pressure and stress determination procedure on the piers is very severe, as stated by the authors. The writer wishes to call attention to the extreme severity involved in the use of the dam analogy for piers surrounded by water. For the large piers of the San Francisco-Oakland Bay Bridge this procedure could well be afforded but this may not be the case with piers of other bridges. Engineers should not feel constrained to apply so severe a test. The writer is of the opinion that because water is present on both sides of the pier the acting forces need not be doubled.

The authors have done a service to the profession by presenting an expanded review of the treatment of earthquake effects in the San Francisco-Oakland Bay Bridge.

NORMAN C. RAAB,[9] M. AM. SOC. C. E., AND HOWARD C. WOOD,[10] ASSOC. M. AM. SOC. C. E. (by letter).—The study of earthquake stresses and effects has been advanced considerably by the valuable discussions of this paper.

Professor Biot has presented the outline of a method that appears to have the important advantage of simplicity of application after the characteristic curve or spectrum has been established. In his conclusions he very properly calls attention to the fact that damping has been neglected. The writers agree that damping can have a considerable effect on stresses and regret that no field measurements are available from which the damping characteristics of the San Francisco-Oakland Bay Bridge might be determined.

It is to be hoped that the necessary data on a number of structures will be assembled and careful research and analytical studies made so that these effects can be evaluated correctly. It seems particularly important to formulate, if possible, some general procedure on damping effects which could be followed during the design stages of a structure, with assurance that it would be reasonably dependable.

Mr. Ulrich has drawn upon his wide experience and studies, making particular use of his extensive knowledge of the El Centro shock of 1940 and of the seismic conditions in the region of the San Francisco-Oakland Bay Bridge. The writers are in accord with his logical and well-grounded conclusions.

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[10] Senior Bridge Engr., San Francisco-Oakland Bay Bridge, Oakland, Calif.

Mr. Moisseiff has discussed very clearly some of the difficulties confronting the engineer engaged in the design of earthquake-resistant structures. While better and more complete information is being accumulated continuously on seismic motion and its effects on structures, the engineer is still faced with the necessity of exercising a large amount of discretion and judgment in the application of the available data to his particular problem. The basic assumptions used in calculating the magnitude and nature of stresses produced by earthquakes are of vital importance in estimating the adequacy of the design of a structure in the so-called "earthquake regions" and will usually have an influence on its cost as well. Fortunately it is found that structures can in many cases be designed to resist earthquake forces without adding materially to the cost of construction.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 2124

SUPERSTRUCTURE OF THEME BUILDING OF NEW YORK WORLD'S FAIR

BY SHORTRIDGE HARDESTY,[1] M. AM. SOC. C. E., AND
ALFRED HEDEFINE,[2] ASSOC. M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. LOUIS BALOG, AND SHORTRIDGE HARDESTY AND ALFRED HEDEFINE.

SYNOPSIS

There are discussed in this paper the unusual engineering problems involved in the design and construction of the Theme Center of the 1939 New York World's Fair. The points of special interest are the spherical steel framework of the Perisphere, the triangular steel tower of the Trylon, and the determination of the wind loads to which these two structures may be subjected.

INTRODUCTION

The Theme Building for the 1939 New York World's Fair, the general features of which are shown in Fig. 1, is composed of four integral parts—the Perisphere, the Trylon, the Bridge, and the Helicline. The Perisphere is the hollow white sphere 180 ft in diameter—as tall as a 16-story building. The Trylon is the needle-like structure—a slender, triangular pyramid taller than the Washington Monument, with its axis 200 ft southeast of the center of the Perisphere. The Bridge is the connecting link between the Perisphere and Trylon, and carries escalator units and walkways. The Helicline is the inclined circular ramp, 950 ft long, which serves as the exit from the group.

The Perisphere, the Trylon, and the Bridge are of steel-frame construction, covered for the most part with a thin stucco covering of a special type, and supported on concrete foundations that rest on creosoted timber piles. The different structures presented individual design problems, some of which were very unusual. The latter statement is particularly true of the Perisphere, which involved the construction of a complete spherical framework, subjected to both vertical loads and transverse wind forces. For its design, the application of the theories of spherical shells was studied thoroughly; but it was

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NOTE.—Published in September, 1940, *Proceedings*.

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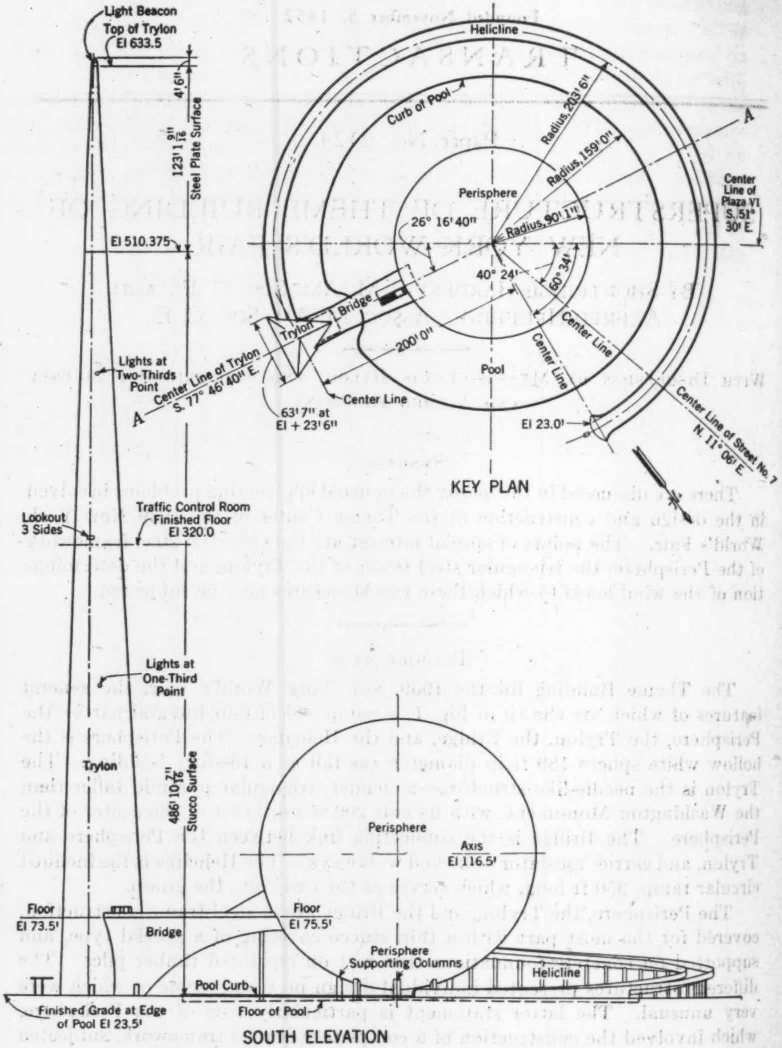
Light Beacon
Top of Trylon El 633.5
4'6"
123'1 9/16"
Steel Plate Surface
El 510.375
Lights at Two-Thirds Point
Lookout 3 Sides
Traffic Control Room Finished Floor El 320.0
Lights at One-Third Point
Trylon
486'10 7/16"
Stucco Surface
Perisphere
Axis El 116.5'
Floor El 73.5'
Bridge
Floor El 75.5'
Perisphere Supporting Columns
Helicline
Pool Curb
Finished Grade at Edge of Pool El 23.5'
Floor of Pool
Helicline
Curb of Pool
Radius 203'6"
A
Radius 159'0"
Center Line of Plaza VI S. 51° 30' E.
Perisphere
26° 16' 40"
Radius 90'1"
40° 24'
60° 34'
Bridge
Trylon
200'0"
Center Line of Trylon S. 77° 46' 40" E.
A
Center Line
Center Line
Center Line
Pool
63'7" at El + 23'6"
El 23.0'
Center Line of Street No. 7 N. 11° 06' E.

SOUTH ELEVATION

KEY PLAN

FIG. 1.—THEME BUILDING, NEW YORK WORLD'S FAIR

eventually found possible, by means of careful reasoning and the visualization of the stress action throughout the frame, to employ the methods applied in the analysis of usual structural frames, in spite of the novelty of the arrangement of the members involved.

The Trylon was unique in that it involved the use of a triangular braced tower so slender that the design was controlled by the wind forces rather than the dead and live loads.

Interesting data regarding the four units of the Theme Building are as follows:

Item No.	Description	Value
	Perisphere	
1	Finished outside diameter, in feet	179.17
2	Finished height above ground, in feet	182.58
	Diameter of Steel Frame, in Feet:	
3	Outer diameter	178.58
4	Inner diameter	162.25
	Meridian Trusses:	
5	Number	32
	Depths, in Feet, at:	
6	Bottom	11.0
7	Equator	8.0
8	Top	5.0
	Girt Trusses:	
9	Number	15
10	Depths, in feet	5 to 10.5
	Ring Girder at Columns (Feet):	
11	Diameter	72.0
12	Depth	7.71
13	Width	2.01
	Columns:	
14	Number	8
15	Height, in feet	16.75
16	Section, in feet	2.65 by 1.88
17	Number of pieces	6,600
18	Number of shop rivets	150,000
19	Number of field rivets	100,000
20	Weight of steel, in pounds	4,300,000
	Covering:	
21	Total area of covering, in square feet	100,851
22	Layers of ½-in. gypsum board (dampproofed between layers and nailed to wooden struts, 2 in. by 4 in.)	2
23	Thickness of stucco, in inches (special magnesite compound)	0.25
	Moving Platforms:	
24	Number (an upper and a lower)	2
	Dimensions, in Feet:	
25	Height above ground (upper and lower)	64, 52
26	Height above Theme exhibit (upper and lower)	30, 18
27	Diameter to outer edge (upper and lower)	113, 103
28	Width	6
29	Distance from inner wall of sphere	12
30	Weight, each platform, in pounds	200,000

Item No.	Description	Value
31	Speed, in feet per minute	60
32	Riding time, in minutes	5.5
33	Estimated spectator load, in pounds	120,000
	Dimensions of Reflecting Pool, in Feet:	
34	Diameter	318
35	Depth	1.5 to 3.5
	Foundations:	
36	Length of piles (528 creosoted Douglas fir), in feet	95 to 104
	Footing Dimensions, in Feet:	
37	Diameter	71
38	Width	14.5
39	Depth	3.5
	Ring Wall Dimensions, in Feet:	
40	Diameter	72
41	Width	4.5
42	Depth	7.75
	Dead Load, in Pounds:	
43	Structural framework	4,300,000
44	Outer shell	720,000
45	Inner shell, insulation, walkways, exhibit, and equipment	1,900,000
46	Moving platforms	400,000
47	Subtotal (items 43–46)	7,320,000
48	Live load, in pounds	480,000
49	Snow load, in pounds	370,000
	Total Load, in Pounds:	
50	8 columns (items 47–49)	8,170,000
51	1 column	1,022,000
	Wind Loads, in Pounds per Column:	
52	Transferred load	270,000
53	Shear	82,000
54	Moment at bottom of column due to wind loads, in pound-feet	656,000
	Trylon	
	Dimensions of Superstructure, in Feet:	
55	Height above ground	610.0
56	Base (length of each side)	63.58
57	Top (length of each side)	2.58
58	Distance of observation room (for traffic control) above ground	296.5
59	Weight of steel, in pounds	1,830,000
	Covering:	
60	Total area, in square feet	60,540
61	Year 1939	
	Layers of gypsum board (see item 22)	1
	Outer surface, magnesite compound stucco	...
62	Year 1940	
	Lower 100 ft stucco	...
	390 ft of plywood painted white	...
63	Height of steel plate construction, in feet, from the top	124
	Foundation Dimensions, in Feet:	
64	Length of piles (513 creosoted Douglas fir)	95 to 104

Item No.	Description	Value
	Hexagonal Bases:	
65	Thickness of two 40-ft bases	7.00
66	Thickness of one 46-ft base	7.75
	Dead Load, in Pounds:	
67	Structural framework	1,830,000
68	Covering	270,000
69	Walls, ceilings, enclosures, anchors	1,200,000
70	Subtotal (items 67–69)	3,300,000
71	Live load, in pounds	110,000
	Total Load, in Pounds:	
72	3 columns (items 70–71)	3,410,000
73	1 column	1,137,000
	Wind Load per Column, in Pounds:	
74	Downward	2,580,000
75	Uplift	2,870,000
	Maximum Load per Column, in Pounds:	
76	Downward	3,717,000
77	Uplift	1,770,000
	Bridge	
	Dimensions of Superstructure, in Feet:	
78	Length	98.5
79	Height at Perisphere and at Trylon	52, 50
80	Radius of soffit line	113.72
81	Length of supporting columns at the Perisphere	49
82	Weight of steel, in pounds	290,000
83	Covering, magnesite compound stucco	....
	Moving Stairways (Two, an Upper and a Lower):	
84	Length, in feet (upper and lower)	120, 96
85	Vertical height, in feet (upper and lower)	60, 48
86	Width, each, in feet	2
87	Angle of rise, in degrees	30
88	Speed, in feet per minute	90
89	Capacity in persons per hour, each	8,000
90	Power (electricity)	....
	Foundations:	
91	Number of piles (creosoted Douglas fir)	32
92	Base dimensions, in feet	23.5 by 10 by 5
	Helicline	
93	Grade (percentage)	5.5
	Dimensions, in Feet:	
94	Length	950
95	Width	18
96	Radius	203.5
	Pipe Column Supports:	
97	Height	18 to 50
98	Diameter	1.5 to 2.5
99	Deck material (timber with stainless steel soffit and cork flooring)	
100	Total cost of Theme Buildings, in dollars	2,000,000

PRELIMINARY DESIGN CONSIDERATIONS: WIND LOADS

Before the engineers began the design of the Perisphere and Trylon, the architects had determined the proportions of the two objects and the general arrangement of the passageways, escalators, moving platforms, and exhibits, and had decided that the Perisphere would be over a circular pool, supported on a row of eight columns arranged in a circle of 72-ft diameter, or on a continuous ring of the same diameter. The column supports were finally chosen, partly for architectural reasons, and partly because wind tunnel tests (described subsequently herein) showed that the wind pressure against the Perisphere would be less for the column supports than for the continuous ring. After the framework of the Perisphere and Trylon had been laid out, the details of the passageways, platforms, etc., were determined, and the resulting loads on the frameworks computed. By assuming the type of cover to be used, the vertical loads, both dead and live, on all parts of the two structures could be calculated.

For the determination of the dead loads, the cover was assumed to be cement stucco, 1 in. thick, weighing 17 lb per sq ft. When a light cover, of a patented magnesite compound, was finally selected in 1939, the Perisphere design was not affected; but the reduction of load on the Trylon required that additional weight be provided to overcome the increased uplift from wind loads. This result was secured by supporting the concrete floor of the Trylon at ground level on heavy concrete beams in which the bottom bracing members of the Trylon framework were embedded.

The foundation materials at the site of the Theme Center consist of about 30 ft of cinders and about 45 ft of mud resting on firm material. In line with other heavy foundations on the Fair grounds, piles driven to the firm material were used; and in order to make it safe to leave the structures standing after the closing of the Fair, if desired, creosoted piles were adopted.

It was obvious that the design of the Perisphere and Trylon would be affected by wind loads; and since structures of these types and sizes had not been constructed before, and information relative to the probable wind action on them was scarce and unsatisfactory, it was decided that a wind-tunnel investigation should be made. This investigation consisted of a series of tests performed under the supervision of Alexander Klemin (Chairman of the Department, Daniel Guggenheim School of Aeronautics, College of Engineering, New York University, New York, N. Y.).[3] The Perisphere model was 2 ft in diameter and was raised 0.08 ft above the ground board; the Trylon model was 6.75 ft high, with a triangular base 0.66 ft on a side. All tests were made in a 9-ft wind tunnel at the laboratory.

The test program included drag and pressure tests on the Trylon and Perisphere, alone and in combination with each other, in various positions relative to the direction of the wind. In order that the "ground effects" experienced by the structures as actually built might be reproduced as closely as possible, the tests of the Perisphere, with and without the Trylon, were made with a "ground board" in the tunnel. The kinematic viscosity was assumed to be 0.000154 ft squared per sec, and the Reynolds number was taken equal to the product of the wind speed in feet per second by the diameter in feet by the constant 6,500 for the case of the Perisphere, and by the length in feet of a side of the base by the same constant for the case of the Trylon. The largest values of the Reynolds number for the wind-tunnel tests, for the cases of the Trylon alone, the Perisphere alone, and the Perisphere and Trylon in combination were 378,000, 572,000, and 1,340,000, respectively.

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[3] "Aerodynamics of the Perisphere and Trylon at World's Fair," by A. Klemin, E. B. Schaefer, and J. G. Beerer, Jr., *Proceedings*, Am. Soc. C. E., May, 1938, p. 887.

Table 1 indicates the values of the pressures on the Perisphere and Trylon for the various conditions tested. It shows the pressure, in pounds per square foot of projected area, for any velocity in miles per hour, together with the values for 100-mile and 90-mile velocities. For reference, the table also includes values for velocity head, and for pressures on large square flat plates, on prisms three times as long as wide, and on indefinitely long prisms or flat plates.

TABLE 1.—PRESSURES IN POUNDS PER SQUARE FOOT OF PROJECTED AREA

Item No.	Description	Formula[a]	Shape factor	PRESSURE[a] FOR:	
				$V = 100$	$V = 90$
1	Velocity head	$0.00256\ V^2$	1.0	25.6	20.8
2	Large, square, flat plate, normal to wind	$0.0032\ V^2$	1.25	32	26
3	Prism 1 by 1 by 3	$0.0038\ V^2$	1.48	38	30.8
4	Infinitely long prism or flat plate	$0.0050\ V^2$	1.95	50	40.5
5	Trylon on the ground: Flat side into wind	$0.0037\ V^2$	1.45	37	30
6	Edge into wind	$0.0023\ V^2$	0.90	23	18.6
7	Perisphere alone, on ring wall, on the ground	$0.0015\ V^2$	0.59	15	12.2
8	Perisphere on ring wall, with Trylon, on ground	$0.0018\ V^2$	0.70	18	14.6
9	Perisphere alone, on columns, on ground	$0.0013\ V^2$	0.51	13	10.5
10	Perisphere on columns, with Trylon, on ground	$0.0016\ V^2$	0.62	16	13
11	Perisphere in free air	$0.0005\ V^2$	0.195	5	4

[a] V = velocity in miles per hour.

Current specifications usually provide that bridges and buildings shall be designed for a maximum wind pressure of 30 lb per sq ft. By referring to Table 1 it will be noted that this pressure corresponds to the value for a large square flat plate at a wind velocity of 100 miles per hr, and also to that for a 1 by 1 by 3 prism in a 90-mile wind.

The "shape factors" determined from the tests on the Trylon and Perisphere, expressed as the ratio of the average wind pressure to the velocity head, are worthy of note. The free Perisphere in an air flow of 90 miles per hr gave an average pressure of 4 lb per sq ft on the projected area, corresponding to a shape factor of 0.195; whereas, when the Perisphere was subject to the same air flow in the presence of the ground, supporting columns, and Trylon, the shape factor increased to 0.62, corresponding to an average pressure of 13 lb per sq ft on the projected area, or an increase in the wind force of three and a quarter times. This phenomenon, caused by the ground and structure effects, indicates the importance of the tests from the structural design point of view.

It is of interest to note that, in the test of the Trylon with a flat side into the wind, there were pressure decreases, or suctions, of sufficient force on those faces away from the direction of the wind to result in a force greater than the velocity pressure times the projected area, corresponding to a shape factor of 1.45. It might also be mentioned that tests with the Trylon directly in front

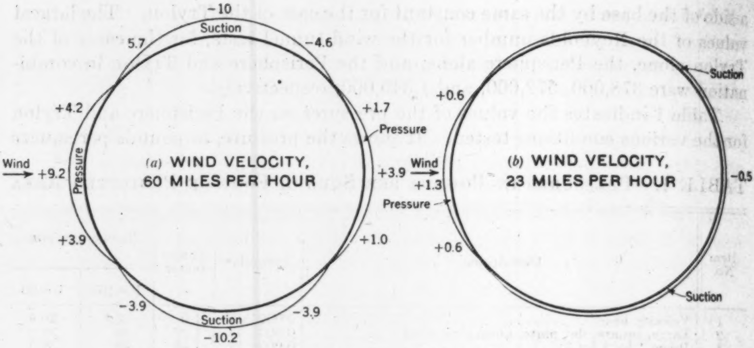
−10
Suction
5.7
−4.6
+4.2
+1.7
Pressure
Wind
+9.2
Pressure
(a) WIND VELOCITY, 60 MILES PER HOUR
+3.9
+1.0
−3.9
−3.9
Suction
−10.2
Suction
+0.6
+3.9
Wind
+1.3
Pressure
(b) WIND VELOCITY, 23 MILES PER HOUR
−0.5
+0.6
Suction

FIG. 2.—WIND PRESSURES ON A SPHERE IN FREE AIR (DIAMETER 9.52 FT)

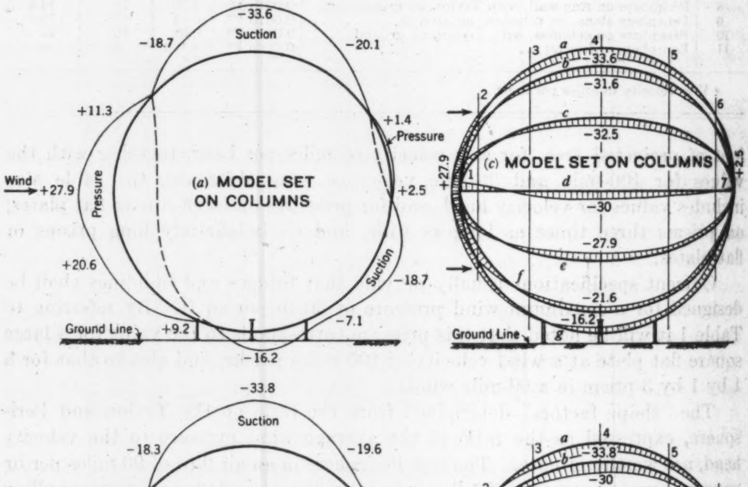
−33.6
Suction
−18.7
−20.1
+11.3
+1.4
Pressure
Wind
+27.9
Pressure
(a) MODEL SET ON COLUMNS
+2.5
+20.6
Suction
−18.7
Ground Line
+9.2
−7.1
−16.2
3
a
4
5
b
−33.6
−31.6
2
6
c
−32.5
(c) MODEL SET ON COLUMNS
+27.9
+2.5
1
d
7
−30
−27.9
e
f
−21.6
−16.2
g
Ground Line
−33.8
Suction
−18.3
−19.6
+11.6
−3.2
Wind
+27.8
Pressure
(b) MODEL SET ON RING WALL
−5.5
+22.9
Suction
−20.8
Ground Line
+20.4
−16.3
3
a
4
5
b
−33.8
−30
2
6
c
−31.3
(d) MODEL SET ON RING WALL
+27.9
−5.5
1
d
7
−27
e
−24
f
−19.6
+20.4
g
−16.3
Ground Line

FIG. 3.—PRESSURES ON A FREE PERISPHERE MODEL DUE TO A WIND VELOCITY OF 100 MILES PER HR

of the Perisphere, blocking the path of the wind, gave a net pressure on the Perisphere of practically nothing.

The values in Table 1 indicate the horizontal resultant of the pressures created by the flow of wind. The drag tests on the Perisphere showed, in addition to the horizontal force, a considerable uplift, amounting to 70% of the horizontal force in the case of the Perisphere alone on ring wall adjacent to the ground, and 60% of the horizontal force in the case of the Perisphere alone on columns adjacent to the ground.

Fig. 2 shows the center meridian pressures, in pounds per square foot, for the supercritical and subcritical states, as determined by O. Flachsbart,[4] at the Göttingen Aerodynamics Institute, on a wood sphere 24.2 cm (9.52 in.) in diameter in free air—that is, with no ground board. Figs. 3(*a*) and 3(*c*) indicate the pressures, in pounds per square foot, on the center meridian of the Perisphere for a wind velocity of 100 miles per hr, for the case of the sphere on columns. The broken lines are isobars of zero pressure. Figs. 3(*b*) and 3(*d*) show graphically the pressure contours for the same model, set on a ring wall. The shaded portion on the outside of a great circle indicates positive pressure, whereas that on the inside of the circle represents a negative pressure, or suction.

The diagrams of Fig. 4 represent the pressures resulting from a wind velocity of 100 miles per hr, based on the various cases tested by Professor Klemin[3] (see Fig. 5). Case I is for that of the Perisphere alone on columns. In this case it is of interest to note that a turbulence was created in the flow of air tailing off the rear of the sphere, thereby forming a back pressure acting against the direction of the wind. Here, as in all cases, the maximum positive pressure occurs on the surface exposed directly to the wind, and the maximum negative pressure on the areas at right angles to the direction of the wind. Case II is for the condition of the Perisphere on columns with the Trylon at its side (with respect to the direction of the wind). This gives essentially the same family of pressure curves as in Case I. The back pressure of Case I, however, has now changed to a slight suction. Case III is for the Perisphere on a ring wall. For Case IV there is the same combination as Case II except that the Perisphere is on a ring wall instead of columns. In both Case III and Case IV there is a considerable build-up of suction at the rear of the sphere, which increases measurably the net force tending to push the Perisphere off of its foundations. Case V is that of the Perisphere on the ring wall with the Trylon directly in front of it with respect to the wind direction. The net result of the pressures for this last case, for all practical purposes, was negligible.

The pressures plotted for each case are those along the seven semi-arcs of great circles which pass through the front and rear poles and whose planes make angles above and below the equator of 0°, 30°, 60°, and 90°. The pressure points are located by increments of 30° azimuth angles through 180° from front to rear. The net wind force acting on the Perisphere for each case tested can be obtained by graphic integration of these pressure curves.

The pressures given in the various diagrams represent the net pressures

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[4] "Recent Researches on the Air Resistance of Spheres," by O. Flachsbart, *Technical Memorandum No. 475*, National Advisory Committee for Aeronautics, 1928.

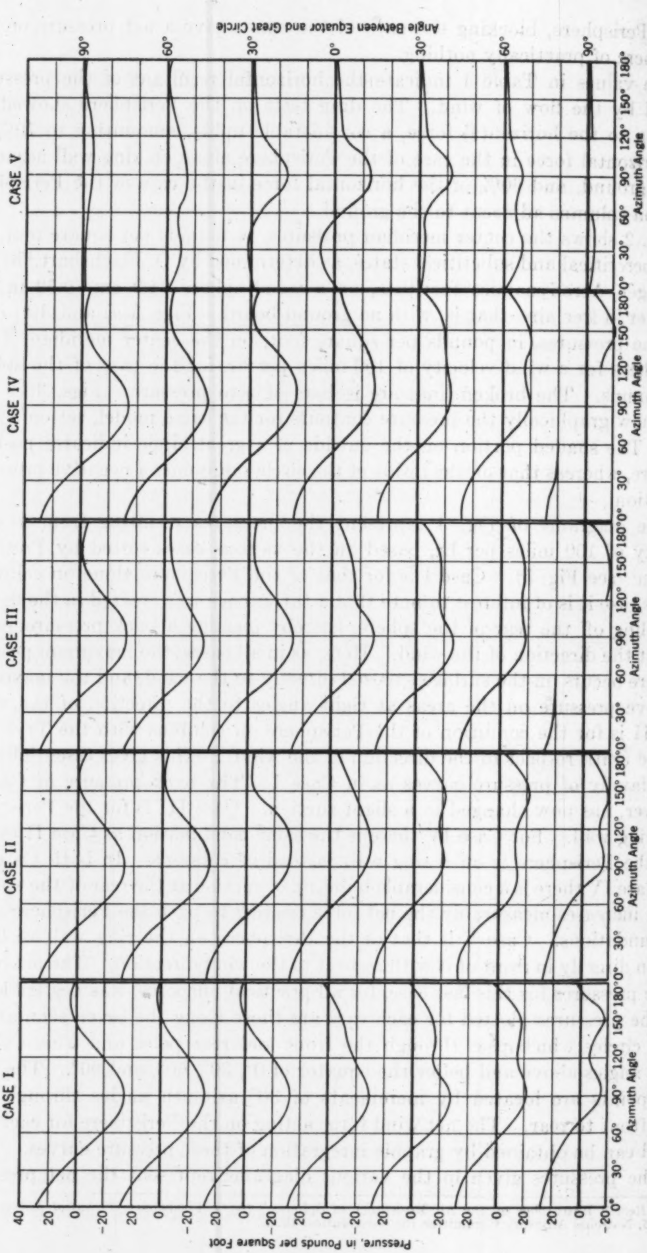


FIG. 4.—PRESSURES ON A PERISPHERE MODEL, WITH TRYLON IN VARIOUS POSITIONS; WIND VELOCITY 100 MILES PER HR

existing on the Perisphere covering at any point when the air pressure within the Perisphere is the same as that in the moving air on the outside. Since the covering is nearly impervious, however, the pressure within may at times be practically that of still air, which is greater than that of moving air by the amount of velocity head. At such times the net pressures on the covering, instead of varying from the velocity head at the portion facing the wind to a suction about one third greater at a point 180° thereto, will vary from about zero at the first point to a suction two and one third times the velocity head at the second point. The suction acting on the cover, therefore, may be as great as 60 lb per sq ft.

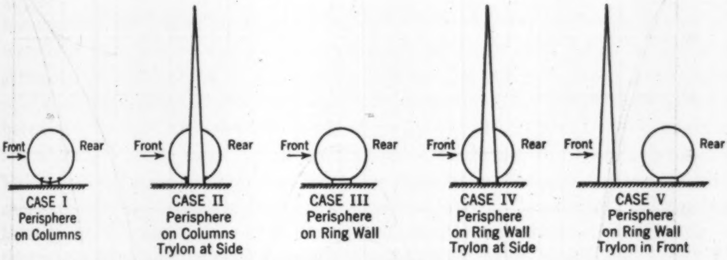


FIG. 5.—POSITIONS OF STRUCTURES WITH RESPECT TO WIND, IN TESTS OF FIG. 4

The curves shown in Fig. 6 present an interesting comparison of wind pressures acting on the Perisphere under various conditions.

As has been previously indicated, there is a large variation in the wind pressures acting over the surfaces of the Perisphere and Trylon. For the Trylon there is positive pressure over the face or faces exposed to the wind, and suction on those away from the direction of the wind, with the maximum pressure approximately equal to the velocity head, and the resultant force acting at a point one third of the way up from the ground surface. In the case of the Perisphere, a bulb of positive pressure occurs over the portion of the surface directly facing the wind, and a high negative pressure or suction occurs over the surface area at 90° to the direction of the wind, reducing to a minimum pressure at a point 180° from the wind direction.

In order to arrive at pressure values for the design of the Perisphere, consideration had to be given to the fact that the presence of the Bridge increased appreciably the forces on the Perisphere itself. The Bridge also causes a slight increase in the wind pressure on the Trylon, but this effect is inappreciable, as the force is applied close to the bottom of the Trylon.

The symmetrical area of suction around the complete Perisphere produces tension in that group of meridian trusses, with very little distortion of the girts. The pressure on the front of the Perisphere, the resultant of which is inclined upward, and the suction on the back, the resultant of which is inclined downward, form a set of forces tending to roll the sphere as a whole.

As the maximum wind velocity in the New York area approximates 90 miles per hr, it was decided to base the design loads on that velocity, with higher values for checking stability against overturning. Therefore, the following loads, in pounds per square foot, were adopted:

Description	For designing members	For determining stability and anchorage
Trylon with flat side into wind	30	50
Trylon with edge into wind	20	33
Perisphere on columns	15	25

Since the completion of the Perisphere and Trylon, they have been subjected to several heavy winds and found to be satisfactory. During the hurricane of September, 1938, a wind velocity of about 75 miles per hr was reached at the site of the Fair. At this time the steel framework of the Trylon, including the steel plate section at the top (but no other outside covering), was completed, and the steel frame of the Perisphere was finished but free of covering or scaffolding. During the storm it was observed that the top of the Trylon moved about 6 in. in either direction from the vertical, which movement compares with the computed movement under full design wind load of about 3 ft.

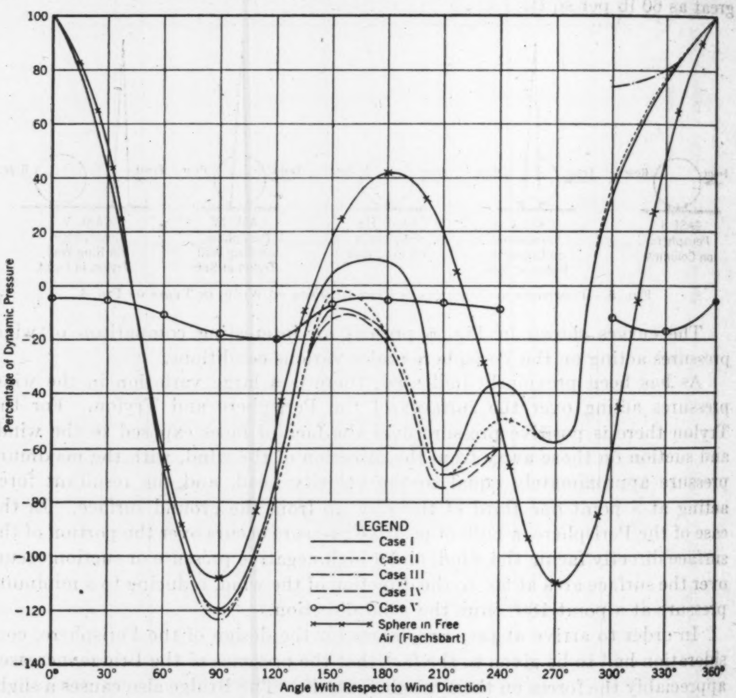


FIG. 6.—PRESSURES ON A MERIDIAN CIRCLE EXPRESSED AS PERCENTAGES OF DYNAMIC PRESSURE

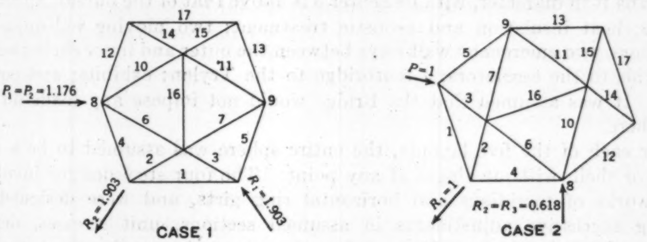
A second observation was made on the Trylon during a subsequent wind which attained a velocity of 45 miles per hr. At that time three fourths of the surface had been covered and a complete scaffolding system shrouded the entire structure. The observed movement at an angle of about 45° to the direction of the wind was about 7 in. on either side of the vertical.

No measurements have been made on the movements of the Perisphere, but after its completion an observer standing at the junction of the Bridge and the Perisphere, with one foot on each structure, noted that only the faintest tremor could be detected during a 45-mile wind.

The framework of the Perisphere was planned to consist of a series of great-circle meridian trusses lying in vertical planes and a series of small-circle horizontal girts, with diagonal bracing in each of the trapezoids formed by the meridians and girts. Analysis showed that this framework, when loaded with symmetrical vertical loads, would have its members subjected to direct stresses of compression and tension, plus secondary bending effects that could be computed, and would transfer the loads directly to the supports. The effect of horizontal forces or unbalanced vertical loads could not be visualized readily, and it was considered desirable to study some other form of framework that could be analyzed directly for such forces and loads. The regular polyhedron having the largest number of faces—the regular icosahedron, consisting of thirty members and twenty triangular faces—was therefore studied.

The icosahedron frame is shown in Table 2. Two loading conditions were considered. In Case 1, the icosahedron is supported at the two ends of one side, and subjected to two horizontal radial forces at mid-height; and in Case 2, it is supported on the three corners of one face, and subjected to a single inclined force. The resulting stresses, which are determinate, are given in the

TABLE 2.—STRESSES IN THE ICOSAHEDRON FRAME



Number	Stress Influence Values		Number	Stress Influence Values		Number	Stress Influence Values		Number	Stress Influence Values	
	Case 1	Case 2		Case 1	Case 2		Case 1	Case 2		Case 1	Case 2
1	−0.138	−0.072	6	−1.100	−0.578	11	−0.292	−0.153	16	+0.361	+0.190
2	+1.421	+0.746	7	+0.516	+0.271	12	−0.043	−0.023	17	−0.138	−0.072
3	−1.198	−0.629	8	+0.642	+0.337	13	−0.043	−0.023	....	....	....
4	+0.458	+0.240	9	+0.362	+0.190	14	+0.112	+0.059	....	....	....
5	−0.543	−0.285	10	−0.292	−0.153	15	+0.112	+0.059	....	....	....

table. By analogy, it was determined that the spherical framework would also transmit transverse forces to the supports by means of direct stresses in the frame members plus secondary bending effects, as in the case of vertical loads.

The icosahedron analogy also afforded a means of estimating the wind stresses in the diagonal bracing. For the Perisphere, the horizontal force of 2 *P* for Case 1 would be about 330,000 lb, which would produce a compression of about 200,000 lb in the diagonals leading down to the leeward support, and a tension of about 220,000 lb in the diagonals leading down to the windward support. Four diagonal members were available to take each of these forces.

Types of Construction Considered

Perisphere.—Preliminary studies for the Perisphere showed that the following types of construction merited consideration: (1) Steel truss framework, separate outer shell; (2) steel beam framework, separate outer shell; (3) steel truss framework, welded steel shell; (4) steel beam framework, welded steel shell; and (5) reinforced concrete shell. The following description of these five types is adapted from reports submitted to the Construction Department of the Fair on April 12, 1937:

Each design provided a sphere of 180 ft outside diameter, carried on eight columns resting on concrete foundations. The columns were to be on a circle of 36-ft radius, the bottom of the sphere being placed 3 ft above the surface of the pool. It was assumed that the outer surface was to be watertight, of accurate outline, as smooth as practicable, and preferably free from visible joints. Provision was made in each design for supporting a lighter inner shell about 164 ft in diameter, with its center 3 ft above that of the outside spherical surface; heat insulation and acoustic treatment; two moving balconies for spectators; two emergency walkways between the outer and inner shells; access platforms to the escalators; a footbridge to the Trylon; exhibits; and equipment. It was assumed that the Bridge would not impose any loads on the Perisphere.

For each of the five layouts, the entire sphere was assumed to be a unit frame or shell, without hinges at any point. The four steel designs involved frameworks of meridians and horizontal ring girts, and were designed by making successive adjustments in assumed sections, unit stresses, deflections, and total stresses. The reinforced concrete layout involved a shell, unstiffened except between the columns, and was designed by means of theories developed for spherical shells.

In the first layout, the framework consisted of trussed members from 5 ft to 11 ft deep, occupying the entire space between the outer and inner shells except at the bottom. There were thirty-two main meridians, spaced about 18 ft apart at the equator. In general, the ring girts were about 16.5 ft on centers. The panels between meridians and rings were braced with X-bracing of single-angle members. The outer shell, which was not intended to take stress, was assumed to be carried by vertical purlins supported by the girts. The inner shell was to be carried by light members supported on the inner flanges of the meridians and girts. The meridians were carried by a circular box girder of 36-ft radius, resting on eight supporting columns. The columns were assumed fixed at their upper ends by the ring girder and the meridians. The design assumed riveted construction throughout, although welded work could be used in certain parts if preferred.

The second layout was similar to the first, except that the members in the upper part of the sphere were curved wide-flanged beams instead of trussed members. Hangers would be required to support the inner shell in this part of the sphere.

The third layout involved a framework similar to that of the first layout, but considerably lighter, with a butt-welded steel outer shell that participated in carrying stress. The diagonal bracing was omitted, as the shell performed its function. The shell was $\frac{5}{16}$ in. thick for the upper parts of the sphere, and reached a maximum thickness of $\frac{7}{16}$ in. in the lower part.

The fourth layout was similar to the third, except that the framework in the upper part consisted of curved beams, as in the case of the second layout.

In the reinforced concrete shell layout, the thicknesses as designed were 3 in. at the top, 4 in. at the equator, and 2 ft 6 in. at the supports. Concrete stresses were low, averaging 200 lb per sq in. except at the supports, where large bending moments occurred. It was assumed that the thin parts of the shell would be of gunite, and the thicker parts of poured concrete. There was ample precedent in European practice for the shell thicknesses adopted, but the requirements of American practice might have called for some modifications; and the conditions at the support were sufficiently unusual to have justified the making of model tests for this part if the reinforced concrete design had been adopted.

The estimated costs of the superstructure and the loads on the foundations for the five layouts were as follows:

Layout	Description	Cost	Total load on foundations, in pounds
1	Steel truss framework, separate outer shell........	$453,000	9,300,000
2	Steel beam framework, separate outer shell..........	401,000	9,000,000
3	Steel truss framework, welded steel shell............	554,000	7,400,000
4	Steel beam framework, welded steel shell............	518,000	7,200,000
5	Reinforced concrete shell........	425,000	15,100,000

The costs of the foundations were to be added in order to determine the comparative total costs.

In comparing the various designs, the following points had to be considered, in addition to the costs:

(1) *Steel Truss Framework, Separate Outer Shell.*—The framework could be fabricated and erected readily by methods in common use, and presented no difficulties other than those arising from the novel shape of the structure. It would provide a stiff, sturdy structure of accurate outline. The outer shell

probably would call for unusual care in construction, if a satisfactory surface were to be obtained. There would be some possibility of objectionable cracks developing as a result of temperature changes or participation in stress action.

(2) *Steel Beam Framework, Separate Outer Shell.*—The comments in item (1) would apply in general to this design. The fabrication should be cheaper than for the first layout, and the erection somewhat more difficult and possibly more expensive. The resulting structure would be entirely satisfactory, although not quite as stiff as in the case of the first layout.

(3) *Steel Truss Framework, Welded Steel Shell.*—The framework members could be fabricated and erected readily. The construction of the shell would call for unusual care in order to secure a satisfactory surface and avoid locked-up temperature stresses. The major difficulty would be to secure even joints; but it was believed that this could be overcome by accurate forming of the plates, and properly controlled welding procedure, with small welding rods. It might be desirable to have tests made to demonstrate the possibilities in this respect. The resulting structure would be somewhat stiffer than that of the first layout; and the shell would be certain to be watertight and free from cracks.

(4) *Steel Beam Framework, Welded Steel Shell.*—The comments in items (1) to (3) would apply in general to item (4), with the modifications noted in connection with layout 2.

(5) *Reinforced Concrete Shell.*—This layout probably would present more construction problems than would the four steel designs, items (1) to (4), but it was believed that they could be solved successfully by companies experienced in concrete construction. The falsework would have to be designed and constructed carefully, in order to secure correct outlines. Accurate workmanship would be required in order to procure a satisfactory outer surface, but this requirement might apply equally to layouts (1) and (2).

After consideration of the construction problems and the relative substructure costs for the various types, it was decided to adopt layout 1, which was found to be no more expensive than the other types, and could be expected to be free from difficult, unforeseen problems that might prove embarrassing. Actual construction has indicated that a correct choice was made.

Trylon.—For the Trylon, four designs were considered, as follows:

Design	Description	Estimated cost of superstructure
1	Braced steel tower construction, surface of thin steel plate	$231,000
2	Braced steel tower construction, gunite or stucco surface	231,000
3	Flat plate construction, consisting of welded steel plate properly stiffened	247,000
4	Reinforced concrete construction	185,000

For designs 1 and 2, the top quarter was to be of flat plate construction similar to design 3.

After taking into consideration the construction problems and the total costs, including foundations, design 2 was adopted.

GEOMETRY AND FRAMEWORK

The outside surface of the completed Perisphere is a true spherical surface, 180 ft in diameter. The inner surface is fundamentally a sphere of 162-ft diameter, with its center 3 ft above the center of the outside sphere; but this surface is modified in the lower part to suit the exhibition and the moving platforms.

The entire Perisphere frame is a unit, of riveted construction throughout, without hinges at any point. The framework consists of curved truss members from 5 ft to 11 ft in depth. There are 32 meridians, each a half of a great circle, intersecting at the top and the bottom, and 15 horizontal girts from 7 ft to 16 ft on centers. The trapezoidal panels formed by the meridians and girts are braced, in the outer surface only, with X-bracing of single-angle members. The intersections of the girts with the meridians lie on small circles; but each section of girt between meridians lies in a great circle. As a result, all meridian and girt members are plane, with all outside chords bent to a uniform radius of 89.29 ft, except for the bottom members, and all inside chords bent to a uniform radius of 81.29 ft. All gusset plates are dished to spherical surfaces. This arrangement of members was evolved by the designers; it is believed to be unique in spherical space structures, and conducive to accurate and satisfactory fabrication. For the girts, it avoided the use of conical girders that would have resulted if each section of girt had been bent to a small circle. The outer sphere was exact in regard to both meridians and girts, whereas the inner sphere was exact in regard to the meridians, but involved slight approximations in regard to the girts.

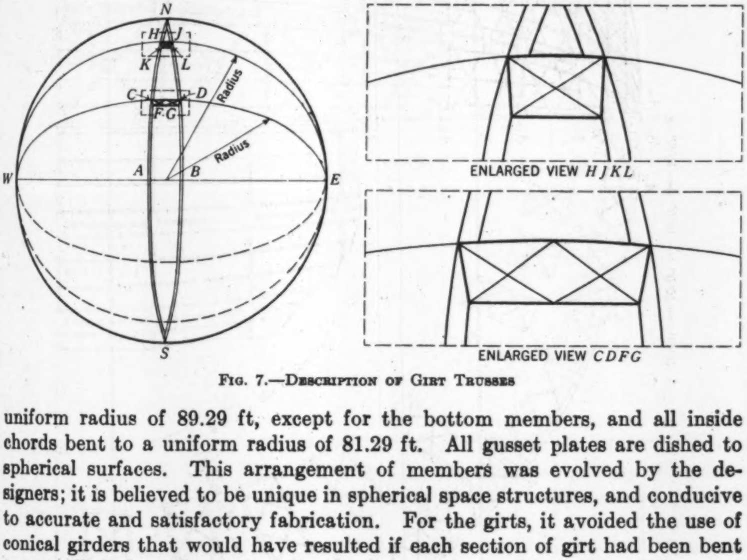
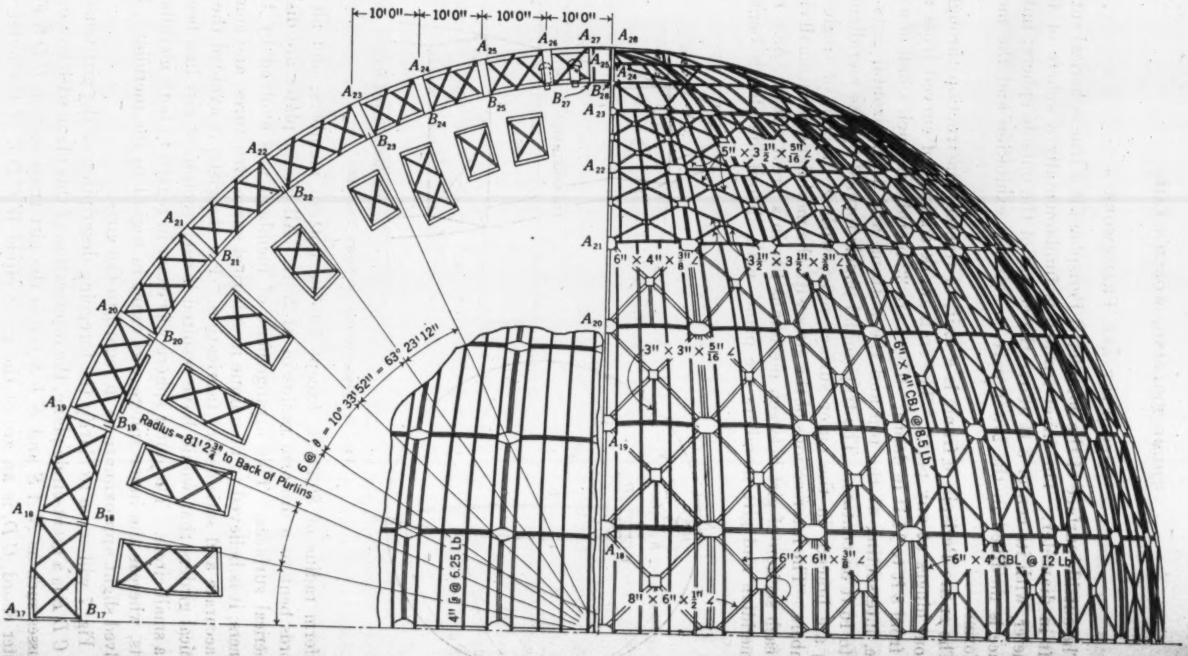
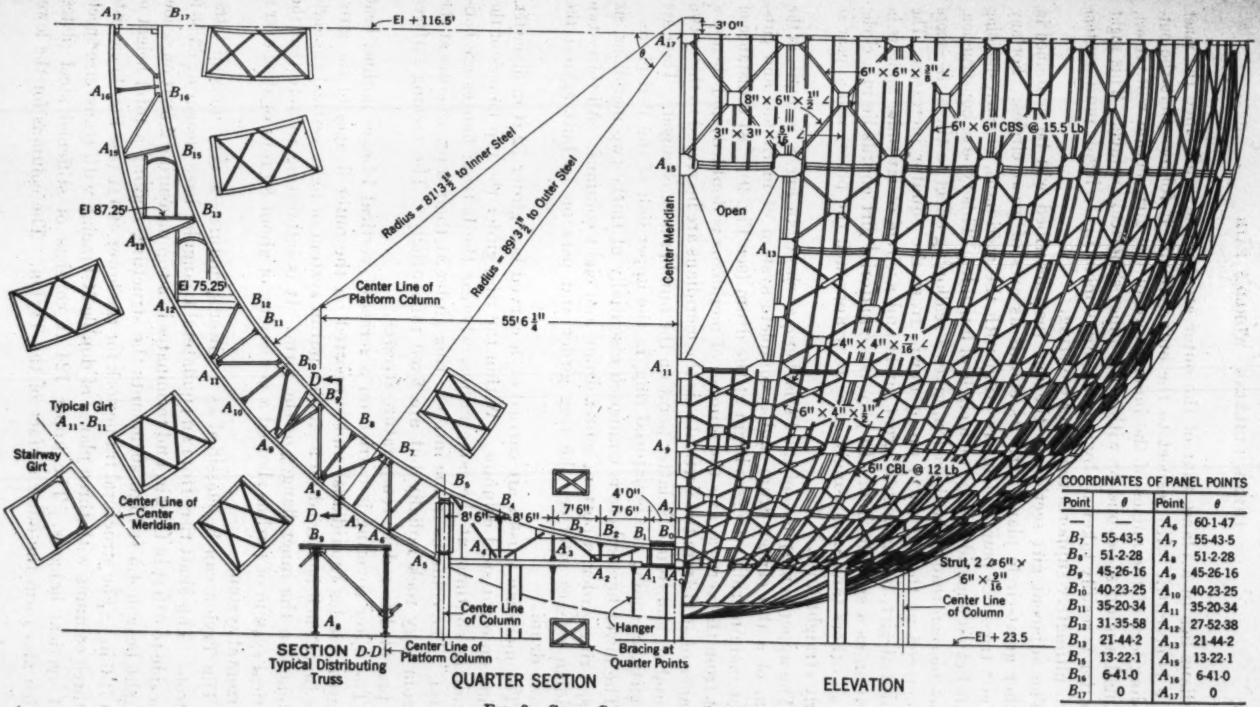


FIG. 7.—DESCRIPTION OF GIRT TRUSSES

Fig. 7 will serve to clarify the foregoing description of the girt trusses. If *W C D E* is a great-circle plane, the intersection of this plane with two meridian trusses such as *N A S* and *N B S* forms the girt truss section *C D G F*. The outer chord, *C D*, is an arc of the great circle *W C D E* of the outer sphere. The inner chord of the girt sections, *F G*, is bent to the radius of 81 ft $3\frac{1}{2}$ in., a great circle arc of the inner sphere. A girt section between any two adjacent


10'0"
10'0"
10'0"
10'0"
A23
A24
A25
A26
A27
A28
A17
A18
A19
A20
A21
A22
B17
B18
B19
B20
B21
B22
B23
B24
B25
B27
B28
Radius = 81'2 3/4" to Back of Purlins
6 @ θ = 10° 33'52" = 63° 23'12"
4" [@ 6.25 Lb
5" × 3 1/2" × 5/16" ∠
6" × 4" × 3/8" ∠
3 1/2" × 3 1/2" × 3/8" ∠
3" × 3" × 5/16" ∠
6" × 4" CBJ @ 8.5 Lb
6" × 6" × 3/8" ∠
8" × 6" × 1/2" ∠
6" × 4" CBL @ 12 Lb


A17
B17
El + 116.5'
3' 0"
θ
A16
B16
A15
B15
El 87.25'
B13
A13
El 75.25'
B12
B11
A12
Radius = 81' 3½" to Inner Steel
Radius = 89' 3½" to Outer Steel
Center Line of Platform Column
55' 6¼"
B10
D
B9
A11
A10
B8
B7
A9
A8
D
A7
A6
A5
B6
B5
B4
A4
8' 6"
8' 6"
7' 6"
7' 6"
4' 0"
B3
B2
B1
B0
A3
A2
A1
A0
Typical Girt
A11 - B11
Stairway Girt
Center Line of Center Meridian
Center Line of Column
Hanger
Bracing at Quarter Points
Center Line of Platform Column
SECTION D-D
Typical Distributing Truss
QUARTER SECTION
Center Meridian
Open
A15
A13
A11
A9
A7
6" × 6" × 3/8" ∠
8" × 6" × 1/2" ∠
3" × 3" × 5/16" ∠
6" × 6" CBS @ 15.5 Lb
4" × 4" × 7/16" ∠
6" × 4" × 1/2" ∠
6" CBL @ 12 Lb
Strut, 2 ∠ 6" × 6" × 9/16"
Center Line of Column
El + 23.5
ELEVATION

COORDINATES OF PANEL POINTS

Point	θ	Point	θ
—	—	A_6	60-1-47
B_7	55-43-5	A_7	55-43-5
B_8	51-2-28	A_8	51-2-28
B_9	45-26-16	A_9	45-26-16
B_{10}	40-23-25	A_{10}	40-23-25
B_{11}	35-20-34	A_{11}	35-20-34
B_{12}	31-35-58	A_{12}	27-52-38
B_{13}	21-44-2	A_{13}	21-44-2
B_{15}	13-22-1	A_{15}	13-22-1
B_{16}	6-41-0	A_{16}	6-41-0
B_{17}	0	A_{17}	0

FIG. 8.—STEEL SKELETON OF PERISPHERE

meridians lies in a plane, that of the outer great circle; and since the inner sphere is eccentric with respect to the outer sphere, it follows that the outstanding legs of the angles of the inner chord of the girt section will project, by small amounts, beyond or within the inner spherical surface. This slight deviation is readily taken up by a minor adjustment in the inside purlin system which ultimately supports the inner covering.

The adjacent girt section to the one just described will be contained in another great-circle plane with its chords as arcs of great circles. For any one girt truss, all points of intersection with the meridians, or corresponding points between meridians, lie on a circle of latitude. This type of construction, as can be seen readily in elevation, gives a scalloping effect to the girts—more pronounced as the distance, up or down, from the equator increases. The equatorial girt, being both a great circle and a circle of latitude, appears in elevation as a straight line. In plan the equatorial girt chords lie in a circle; but all others (and increasingly so as they approach the poles) will appear as nearly straight lines.

The upper hemisphere of the Perisphere, as shown in Fig. 8, is made, in the main, of a top center drum and a complete system of meridians and girts. Eight main meridian trusses begin at the drum (see Fig. 9). At a distance of 20 ft from the vertical axis the number of meridians is doubled; and then at a distance of 40 ft from the vertical axis the meridians are increased to thirty-two in number; and from this latter point on the number is unchanged. There are ten girts, including the equatorial ring, in the upper half of the Perisphere.

The lower hemisphere is composed essentially of thirty-two meridians, six girts, a ring girder, a bottom center drum, and eight columns. All thirty-two meridian trusses intersect the ring girder and pass on to butt against the bottom drum.

The meridian trusses are carried on a curved box girder 72 ft in diameter, resting on eight steel columns. Within the ring girder the top flanges of the meridians continue at the same radius, whereas the bottom flanges are horizontal. The columns frame into the ring girder at their tops, whereas at the bottom they rock radially but are fixed tangentially. The rocking surfaces are provided to reduce temperature stresses.

The outer covering is carried on a series of vertical I-beam purlins, bent to great circles (meridians) and supported on the outer flanges of the girts. The inner shell is carried on a similar purlin system of light vertical channels fastened to the inner flanges of the girts. It is interesting to note that the vertical system of purlins showed a net saving of about 80 tons of steel over a horizontal system.

The Trylon can be classified as a tower structure—a very unusual tetrahedron. This shaft rises from an equilateral triangular base measuring 63.58 ft on a side to 610 ft in the air, and terminates in a top measuring 2.58 ft on a side. A light beacon 4.5 ft high surmounts the structure, making a total height of 614 ft 6 in. The structural framework for the lower 490 ft (see Fig. 1) consists of three columns with three planes of double X-bracing with transverse sheets and vertical hangers. The upper 124 ft consists of stiffened steel plates, which also form the outer surface of the Trylon. The columns for the lower

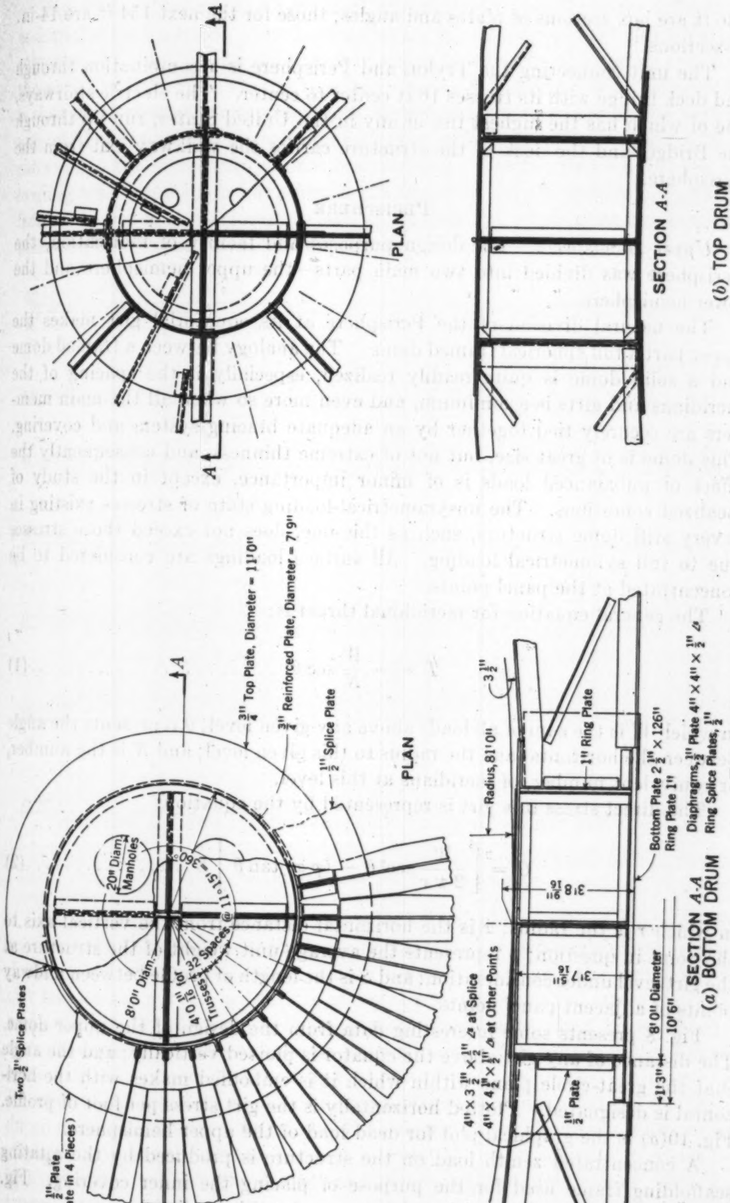

A
A
PLAN
SECTION A-A
(b) TOP DRUM
A
A
$\frac{3}{4}''$ Top Plate, Diameter = 11'0"
$\frac{3}{4}''$ Reinforced Plate, Diameter = 7'9"
$1\frac{1}{2}''$ Splice Plate
PLAN
Two $\frac{1}{2}''$ Splice Plates
20" Diam. Manholes
8'0" Diam.
$4'0\frac{1}{16}''$ for 32 Trusses
32 Spaces @ 11°15' = 360°
$\frac{1}{2}''$ Plate (Fabricate in 4 Pieces)
$3\frac{1}{2}''$
Radius, 81'0"
1" Ring Plate
$3'8\frac{9}{16}''$
$3'7\frac{5}{16}''$
Bottom Plate $2\frac{1}{2}'' \times 126''$ Ring Plate 1"
Diaphragms, $\frac{1}{2}''$ Plate $4'' \times 4'' \times \frac{1}{2}''$ ∠
Ring Splice Plates, $\frac{1}{2}''$
8'0" Diameter
10'6"
1'3"
$\frac{1}{2}''$ Plate
$4'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ ∠ at Splice
$4'' \times 4\frac{1}{2}'' \times \frac{1}{2}''$ ∠ at Other Points
SECTION A-A
(a) BOTTOM DRUM

FIG. 9.—FRAMING DETAIL OF DRUMS

336 ft are box sections of plates and angles; those for the next 154 ft are 14-in. H-sections.

The unit connecting the Trylon and Perisphere is a combination through and deck bridge with its trusses 16 ft center to center. The electric stairways, one of which has the highest rise of any in the United States, run up through the Bridge, and the deck of the structure carries the walkway exit from the Perisphere.

PERISPHERE

Upper Hemisphere.—For design purposes and facility of fabrication, the Perisphere was divided into two main parts—the upper hemisphere and the lower hemisphere.

The natural division of the Perisphere at its equatorial girt makes the upper part a full spherical framed dome. The analogy between a framed dome and a solid dome is quite readily realized, especially if the spacing of the meridians and girts is a minimum, and even more so when all the main members are securely tied together by an adequate bracing system and covering. This dome is of great size, but not of extreme thinness, and consequently the effect of unbalanced loads is of minor importance, except in the study of localized conditions. The unsymmetrical-loading state of stresses existing in a very stiff dome structure, such as this one, does not exceed those stresses due to full symmetrical loading. All surface loadings are considered to be concentrated at the panel points.

The general equation for meridional thrust is:

$$T = -\frac{W}{N} \sec\theta \quad \text{.......................(1)}$$

in which W is the sum of all loads above any given level; θ represents the angle between the horizontal and the radius to this given level; and N is the number, or equivalent number, of meridians at this level.

The direct stress in a girt is represented by the equation:

$$G = \left[\frac{W}{2\pi r}\sec^2\theta - (p\,x)\tan\theta\right] S \quad \text{...............(2)}$$

in which r is the radius; x is the horizontal distance from the vertical axis to the point in question; p represents the average unit weight of the structure at the girt level under consideration; and S is the length of profile between midway points of adjacent panel points.

Fig. 8 presents some interesting data from the design of the upper dome. The distance of any girt above the equator is plotted vertically; and the angle that the great-circle plane within which it is embodied makes with the horizontal is designated. Plotted horizontally is the girt stress per foot of profile. Fig. 10(*a*) is the graphical plot for dead load of the upper hemisphere.

A concentrated zenith load on the structure is produced by the rotating scaffolding frame used for the purpose of placing the inner covering. Fig. 10(*b*) is the plot of the resulting girt stresses for this loading.

Fig. 10(*c*) represents three different sets of girt stresses for three differently assumed snow loadings. The loaded surface for all cases is that of a spherical segment 26 ft high. The uniform load curve is drawn on the assumption that the entire surface is subjected to a 25-lb-per-sq-ft loading. This assumption makes no allowance for the fact that, as the surfaces recede from the vertical axis of the Perisphere, they become steeper and hence cannot retain a 25-lb snow as do the more nearly horizontal surfaces. For the two other cases, the loading was assumed to have first a circular, and then a parabolic, variation from 25 lb per sq ft maximum to zero as a minimum. The latter two cases are perhaps closer to the actual load experienced by the structure than the first case.

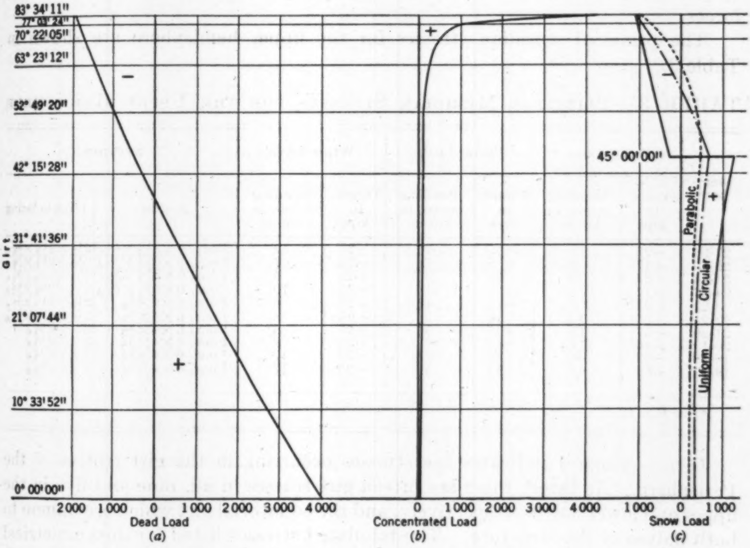


FIG. 10.—GIRT STRESS PER FOOT OF PROFILE

All meridians and girts for the upper hemisphere, except the diagonal meridians and the girts near the top, are comprised of two-angle chord sections double laced with single angles. Since the chords of both the meridian and girt trusses are curved, a bending moment equal to the axial stress times the mid-ordinate of the curve between lacing intersections must be considered; and, since the girt chords act also as supporting members for the vertical purlin system, the unsymmetrical bending that occurs produces another set of stresses. In order to reduce the amount of unsymmetrical bending, a hanger was introduced from the intersection of the X-bracing. Since the bracing angles are straight from panel point to panel point, and the outer chord of the girt lies in a spherical surface, the hanger is skewed and produces a kick upon the trussed girt chord.

The center drum of the upper half of the Perisphere (see Fig. 9) consists of a flat ½-in. plate 9 ft 3 in. in diameter acting as a splicing medium for the top of the eight meridians that attach to the drum as well as a top plate of the drum itself. The bottom plate of the drum is ½ in. thick and 5 ft 8 in. in diameter. A ½-in. ring splice plate is used for fixing the bottom chords of the eight meridians to the drum. The vertical ring of the drum is comprised of a ¾-in. plate 4 ft 9 in. deep and two angles 4 in. by 4 in. by ½ in. bent to conform to a diameter of 5 ft 10 in. The meridian trusses attach to this vertical ring plate by means of 8-in. by 4-in. by ½-in. angles. This detail, further aided by the two vertical cross diaphragms, made up of ½-in. plates and 4-in. by 4-in. by ½-in. angles, forms a rigid unit capable of transmitting the shearing forces.

The principal meridian stresses for the upper hemisphere are shown in Table 3.

TABLE 3.—PRINCIPAL MERIDIAN STRESSES FOR THE UPPER HEMISPHERE

Member[a]	DEAD LOAD		SNOW LOAD		WIND LOAD		SECTION	
	Direct, in kips	Bending stress, in kip-in.	Direct, in kips	Bending stress, in kip-in.	Direct, in kips	Bending stress, in kip-in.	Angles	Double lacing
27-26	−25	13	− 5	2	−25	12	Four, $6\times3\frac{1}{2}\times\frac{5}{16}$	$2\frac{1}{2}\times2\frac{1}{2}\times\frac{5}{16}$
26-25	−17	4	− 8	4	−25	7	Four, $6\times3\frac{1}{2}\times\frac{5}{16}$	$2\frac{1}{2}\times2\frac{1}{2}\times\frac{5}{16}$
25-24	−25	6	−12	3	−25	6	Four, $6\times3\frac{1}{2}\times\frac{5}{16}$	$2\frac{1}{2}\times2\frac{1}{2}\times\frac{5}{16}$
24-23	−17	4	− 9	4	−25	10	Four, $6\times3\frac{1}{2}\times\frac{5}{16}$	$2\frac{1}{2}\times2\frac{1}{2}\times\frac{5}{16}$
23-22	−24	7	−12	4	−25	6	Four, $6\times4\times\frac{3}{8}$	$2\frac{1}{2}\times2\frac{1}{2}\times\frac{5}{16}$
22-21	−30	10	−16	5	−25	8	Four, $6\times4\times\frac{3}{8}$	$2\frac{1}{2}\times2\frac{1}{2}\times\frac{5}{16}$
21-20	−37	12	−15	5	−25	8	Four, $6\times4\times\frac{3}{8}$	$3\times3\times\frac{5}{16}$
20-19	−46	14	−13	4	−25	8	Four, $6\times4\times\frac{3}{8}$	$3\times3\times\frac{5}{16}$
19-18	−53	36	−12	8	−25	17	Four, $6\times4\times\frac{3}{8}$	$3\times3\times\frac{5}{16}$
18-17	−64	43	−12	8	−25	17	Four, $6\times4\times\frac{3}{8}$	$3\times3\times\frac{5}{16}$

[a] See Fig. 12.

Girts.—Table 4 indicates the stresses occurring in the girt trusses of the Perisphere. As listed, there are fifteen girt trusses in all, nine and five in the upper and lower halves, respectively, and the equatorial girt which is common to both halves of the structure. The tabulated stresses listed for unsymmetrical bending are those for the outer chords of the girts only. In girts 17 to 7, inclusive, the stresses resulting from snow load are included in the values listed under the dead-load columns. As will be noted, for main action, the girt truss at panel points 13 is completely idle and the one at panel point 7 partly so. In Fig. 8 is shown the one opening in the sphere, approximately 34 ft square, into which projects, without imposing any load on the Perisphere, the escalator and walkway unit—the Bridge. Being two panels in height, this aperture breaks the continuity of girt truss 13. As a result of this interruption, the lower half of the Perisphere was structurally designed as if girt 13 had been omitted; at the same time the girt was necessary at this location to carry the purlin systems, platforms, and coverings. The requirement that the structure be allowed to deform as designed, without any restraining action on the part of this discontinuous girt, was accomplished by making its panel-point connections with ⅞-in. bolts in 1$\frac{1}{16}$-in. holes, the nuts on the bolts being drawn up lightly and the threads checked. In adhering to the analysis of the lower hemisphere, and in order to control more closely the B_8 point, there must be no force exerted at the inner panel point of girt truss 7. Hence, these panel points were treated in the same manner as those of girt truss 13. The outer panel point of girt 7 has little or negligible dead-load stress, and as a result no control over the B_8 point; consequently it was riveted tight.

Girt 9 has an unsymmetrical inner flange whose section is comprised of an 8-in. by 6-in. angle and a 6-in. by 6-in. angle. The area provided by this inner flange is in excess of that required for stress, but was made necessary in order to accommodate the proper seating, connections, and framing for the bases of the moving platform columns.

TABLE 4.—STRESSES IN PERISPHERE GIRT TRUSSES

Girt member at panel point:	DEAD LOAD			SNOW LOAD			WIND LOAD		SECTION	
	Direct, in kips*	Bending Moments, in Kip*-In.		Direct, in kips*	Bending Moments, in Kip*-In.		Direct, in kips*	Bending moment, in kip*-in.	Angles	Double lacing
		Direct	Unsymmetrical		Direct	Unsymmetrical				
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
26	− 20	9	28	−11	5	23	− 7	31	Two, 6×4×3/8	3×3×5/16
25	− 19	10	28	−11	6	23	− 8	33	Two, 6×4×3/8	3×3×5/16
24	− 17	6	14	−10	4	14	−11	17	Two, 6×6×3/8	3×3×5/16
23	− 20	11	31	−11	6	26	−11	35	Two, 6×6×3/8	3×3×5/16
22	− 17	6	11	− 9	3	10	−15	13	Four, 6×3½×3/8	3×3×5/16
21	− 6	2	23	+20	7	22	−20	20	Four, 6×3½×5/8	3×3×5/16
20	+ 9	4	37	+15	6	....	−20	28	Four, 6×4×3/8	3×3×5/16
19	+ 26	17	37	+14	8	....	−22	24	Four, 6×4×3/8	3×3×5/16
18	+ 44	32	30	+12	8	....	−25	34	Four, 6×4×7/16	3×3×5/16
17	+164	46	30	....	....	....	−20	60	Four, 6×6×1/2	3×3×5/16
15	+190	94	41	....	....	....	+42	80	Four, 6×6×9/16	3½×3×5/16
13	....	....	26	....	....	....	....	....	Four, 6×6×1/2	3½×3×5/16
11	+328	112	21	....	....	....	+35	+41	Two, 8×8×3/4 Two, 8×8×7/8	4×3×5/16
9	+109	34	16				− 8	32	One, 8×6×9/16 One, 6×6×7/16 Two, 6×4×9/16	4×3×5/16
7	....	....	10				+21	42	Four, 6×4×7/16	4×3×5/16

* 1 kip ("kilo-pound") = 1,000 lb

Lower Hemisphere.—The lower hemisphere of the Perisphere was designed by a process of trial and error, involving a series of converging adjustments in sectional areas, unit stresses, deformations, and total stresses. Each meridian truss below the equator was assumed to be a cantilever truss supported vertically on the ring girder and fixed horizontally at the vertical axis of the sphere; loaded at its upper end with the vertical loads from the upper hemisphere and at its various panel points with the vertical loads of the steelwork, inner and outer covering, circular platforms, equipment, and live loads; and restrained against outward movement by horizontal pulls from the various girts. The vertical loads were known, or could be assumed with sufficient accuracy. As a first approximation, a set of horizontal forces from the girts was assumed that was just large enough to balance the moment of the vertical forces. The stresses in the various members of the cantilever truss were then computed, sectional areas assumed, and the outward deflections of the various panel points of the truss computed by means of a Williot diagram. From the outward movement at the level of each girt, the resulting unit stresses in the girt could be computed. Since the first set of girt pulls had been assumed arbitrarily, the resultant horizontal displacements and girt stresses were not satisfactory, the cantilever trusses being bent considerably out of shape, and some of the girts showing very high tension and others considerable compression. After an examination of the results, a new set of horizontal pulls was assumed; the stresses, sections, deformations, and deflections of the cantilever truss recomputed; and the

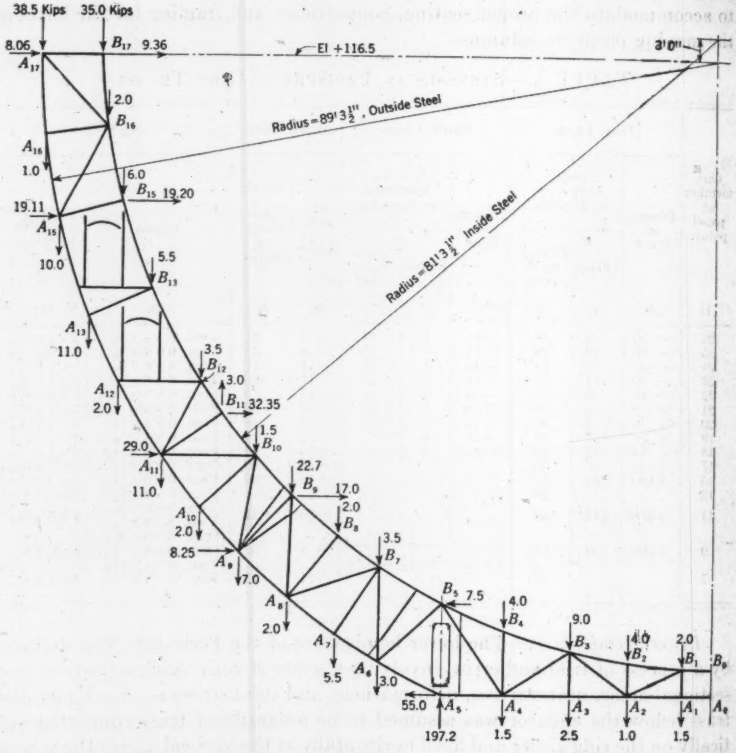

38.5 Kips
35.0 Kips
8.06
A_{17}
B_{17}
9.36
El +116.5
3'0"
2.0
B_{16}
Radius = 89' 3½", Outside Steel
A_{16}
1.0
6.0
B_{15}
19.20
19.11
A_{15}
10.0
5.5
B_{13}
Radius = 81' 3½" Inside Steel
A_{13}
11.0
3.5
B_{12}
3.0
B_{11}
32.35
A_{12}
2.0
1.5
B_{10}
29.0
A_{11}
11.0
22.7
B_9
17.0
2.0
B_8
A_{10}
2.0
8.25
A_9
7.0
3.5
B_7
A_8
2.0
B_5
7.5
4.0
B_4
9.0
B_3
4.0
B_2
2.0
B_1
B_0
A_7
5.5
A_6
3.0
55.0
A_5
A_4
A_3
A_2
A_1
A_0
197.2
1.5
2.5
1.0
1.5

FIG. 11.—LOADING DIAGRAM; FINAL LOADS AND RING PULLS

resultant girt unit stresses again computed. After a number of successive trials, it was found possible to arrive at a series of horizontal pulls that would give reasonably uniform unit stresses of the desired intensity in a set of girts of moderate cross-sectional areas. It was found further that if any attempt were made to vary appreciably the various horizontal pulls thus determined, the cantilever truss would be forced considerably out of shape, and a balance between girt pulls, girt unit stresses, and girt sections could not be reached. In other words, it was found that the girt stresses could not differ appreciably from those arrived at. A change of as little as 100 lb in the horizontal pull applied by a given girt would throw the cantilever truss appreciably out of shape. It was also found that the cantilever trusses would be very rigid, since the girts would permit only very small transverse movements.

The action of the steel framework of the Perisphere under the dead, live, and snow loads is one in which the meridians, in compression, tend to bow out; this tendency in turn brings the girts into play as tension rings. One can have a very clear mental picture of the entire structural action by pressing uniformly on top of a rubber ball set in a ring collar about one third the diameter of the ball. In reacting to these dead, live, and snow loads the Perisphere will deform from the outline of a true sphere. When it is subjected to final loads and ring pulls as indicated in Fig. 11, the Perisphere will deform so as to allow a point on the equator to move radially 0.25 in. and vertically 0.5 in. The principal meridian stresses for the lower hemispheres are shown in Table 5.

TABLE 5.—PRINCIPAL MERIDIAN STRESSES FOR THE LOWER HEMISPHERE

(a) OUTSIDE STEEL							(b) INSIDE STEEL			
			Section							
			Typical		Entrance					
Member[a] (1)	Direct load,[b] in kips (2)	Bending stress,[b] in kip-in. (3)	Two angles (4)	One plate (5)	Two angles (6)	One plate (7)	Member[a] (1)	Direct load,[b] in kips (2)	Bending stress,[b] in kip-in. (3)	Typical section (two angles) (4)
A17-A15	−27	50	6×6×7/16	....	....	....	B17-B16	−38	63	6×6×7/16
A15-A11	−25	56	8×6×9/16	....	....	....	B16-B15	−59	100	6×6×9/16
A11-A9	−53	56	6×6×7/16	7×5/8	6×6×1/2	7×5/8	B15-B12	−103	257	8×6×9/16
A9 -A8	−65	86	6×6×7/16	7×5/8	6×6×1/2	7×5/8	B12-B10	−89	46	8×6×9/16
A8 -A7	−113	97	8×6×9/16	7×5/8	8×6×11/16	7×5/8	B10-B9	−122	114	8×6×3/4
A7 -A6	−117	97	8×6×9/16	7×3/4	8×6×11/16	7×3/4	B9 -B7	−127	145	8×6×3/4
A6 -A5	−209	....	8×8×13/16	....	....	....	B7 -B5	−51	93	8×8×13/16
A5 -A4	−132	....	8×8×3/4	6½×3/4	8×8×7/8[c]	....	B5 -B3	+40	62	8×8×3/4
A4 -A2	−101	....	8×8×3/4	....	....	....	B3 -B1	−5	....	8×8×3/4
A2 -A1	−70	....	8×8×3/4	....	....	....	....	....	....	....

[a] See Fig. 12. [b] Dead load plus live load. [c] At columns.

The emergency exit fire passageways contained between the outer and inner shells of the Perisphere at the 75.5-ft and 87.5-ft levels necessitated the plate section of meridian truss between the 12th and 15th panels.

In Table 5(*a*) (lower hemisphere) it will be noted that the sections of the outer meridian chords between panel points 11 and 6 are not the usual two-angle section, but have plates 7 in. deep placed between the angles and extending from gusset plate to gusset plate, but not developed into the joints. This type of member affords ample chord section for direct stress in the panel point region as well as for the increased stress due to bending at the mid-panel point. Table 5 (lower hemisphere) indicates heavier chord members below panel point 9 at the entrance side of the Perisphere, which was done to accommodate the increase in stresses due to the extra dead and live loads concentrated at the junction of the Perisphere and Bridge. The net unbalanced steel load of 50,000 lb at the entrance to the Perisphere is partly counteracted by placing,

advantageously, some of the heavier electrical equipment in the four bays diametrically opposite the entrance. Another aid in balancing the extra load at the entrance is the placing of a concrete carrying slab for the electrical apparatus as compared with the lighter composition floor on the entrance platforms.

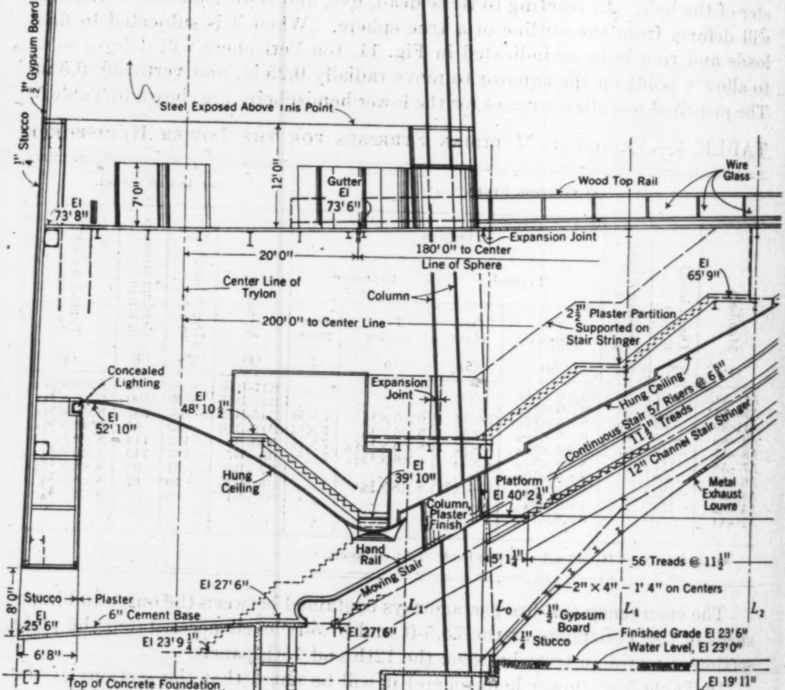
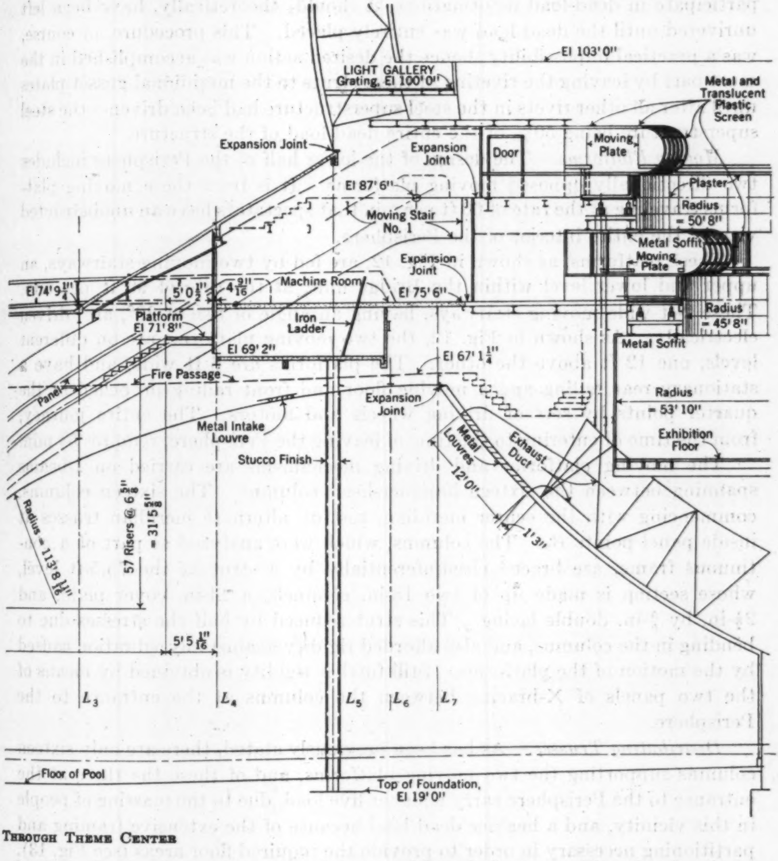

1/2" Gypsum Board
1/4" Stucco
Steel Exposed Above This Point
El 73' 8"
7' 0"
12' 0"
Gutter El 73' 6"
Wood Top Rail
Wire Glass
Expansion Joint
20' 0"
180' 0" to Center Line of Sphere
El 65' 9"
Center Line of Trylon
Column
2 1/2" Plaster Partition Supported on Stair Stringer
200' 0" to Center Line
Concealed Lighting
El 48' 10 1/2"
El 52' 10"
Expansion Joint
Hung Ceiling
Continuous Stair 57 Risers @ 6 5/8"
11 1/2" Treads
12" Channel Stair Stringer
Hung Ceiling
El 39' 10"
Platform El 40' 2 1/4"
Column, Plaster Finish
Metal Exhaust Louvre
Hand Rail
5' 1 1/4"
56 Treads @ 11 1/2"
8' 0"
Moving Stair
El 27' 6"
2" × 4" – 1' 4" on Centers
Stucco
Plaster
El 25' 6"
6" Cement Base
L_1
L_0
L_1
L_2
1/2" Gypsum Board
1/4" Stucco
El 23' 9 1/4"
El 27' 6"
Finished Grade El 23' 6"
Water Level, El 23' 0"
6' 8"
El 19' 11"
Top of Concrete Foundation of Trylon, El 17' 6"
Trylon Foundation, El 17' 6"

FIG. 12.—GENERAL SECTION

In the final analysis, due to the nature of the structure (that is, its geometrical arrangement), the Perisphere framework was found to be extremely sensitive to even small changes in the assumed distribution of stresses in the various parts. This supersensitive characteristic of the structure is a measure of design safety. Since the entire framework strives to distribute quickly all types of loadings, it is of prime importance that very careful study be accorded the connections and framing of all members. Inasmuch as the structure was

designed to conform to particular deformations, resulting in specified sets of stresses, care was taken to avoid, as far as it was physically possible, any interference by members or their connections with the desired deformations of the structure.

The ring girder as a member receives its stress from bending due to vertical


El 103'0"
LIGHT GALLERY
Grating, El 100'0"
Metal and Translucent Plastic Screen
Expansion Joint
Expansion Joint
Door
Moving Plate
El 87'6"
Plaster
Moving Stair No. 1
Radius = 50'8"
Metal Soffit
Moving Plate
Expansion Joint
El 74'9 1/4"
5'0 1/2"
4 9/16"
Machine Room
El 75'6"
Radius = 45'8"
Platform El 71'8"
Iron Ladder
Metal Soffit
El 69'2"
El 67'1 1/4"
Fire Passage
Panel
Expansion Joint
Radius = 53'10"
Metal Intake Louvre
Metal Louvres
Exhaust Duct
Exhibition Floor
Stucco Finish
12'0"
11'3"
57 Risers @ 6 5/8" = 31'5 5/8"
Radius = 113'8 11/16"
5'5 1/16"
L3
L4
L5
L6
L7
Floor of Pool
Top of Foundation, El 19'0"

THROUGH THEME CENTER

loads and ring shortening caused by meridional thrusts. The chords of the meridian trusses pass above and below the flanges of the ring girder. This arrangement facilitated fabrication and erection but complicated the connection for transferring meridional thrusts into the ring girder; and a further complication which makes this particular connection of the utmost importance is the abrupt change in the path of the outer chord stresses caused by the outer meridional chords turning horizontally at this point.

In the outer cross bracing system, just outside the ring girder, there is a two-angle wind strut connected to the bottom cover plates of the ring girder by ⅝-in. bent plates. By means of this device the wind shear is transferred into the bottom girder flange. In order that this strut, which is in reality an auxiliary ring flange, shall be reserved for wind action only and shall not participate in dead-load deformations, it should, theoretically, have been left unriveted until the dead load was entirely placed. This procedure, of course, was a practical impossibility; hence the desired action was accomplished in the most part by leaving the riveting of these struts to the meridional gusset plates until after all other rivets in the steel superstructure had been driven—the steel superstructure being 50% of the entire dead load of the structure.

Moving Platforms.—The design of the lower half of the Perisphere includes two directionally opposed moving platforms. It is from these moving platforms traveling at the rate of 60 ft per min that spectators have an unobstructed view of the entire interior of the Perisphere.

These platforms, as shown in Fig. 12, are fed by two moving stairways, an upper and lower level within the Bridge unit, at the rate of 90 ft per min. These 2-ft wide moving stairways, having an angle of rise of 30°, are driven electrically. As shown in Fig. 12, the two moving platforms are on different levels, one 12 ft above the other. The platforms are 6 ft wide, and have a stationary rear railing and a moving floor and front railing propelled at the quarter points by sets of driving wheels and motors. The entire journey, from the time of entering to the time of leaving the Perisphere, requires 5½ min.

The moving platforms and driving mechanisms are carried on I-beams spanning between the sixteen hammer-head columns. The sixteen columns, commencing with the center meridian, rest on alternate meridian trusses at inside panel points B_9. The columns, which were analyzed as part of a continuous frame, are braced circumferentially by a strut at the 75.5-ft level, whose section is made up of two 18-in. channels, a 14-in. cover plate, and 2½-in. by ⅜-in. double lacing. This strut reduced by half the stresses due to bending in the columns, and also afforded rigidity against any vibration caused by the motion of the platforms. Still further rigidity is obtained by means of the two panels of X-bracing between the columns at the entrance to the Perisphere.

Distributing Trusses.—As has been previously stated, there are only sixteen columns supporting the two moving platforms, and of these the three at the entrance to the Perisphere carry a higher live load, due to the massing of people in this vicinity, and a heavier dead load because of the extensive framing and partitioning necessary in order to provide the required floor areas (see Fig. 13).

These sixteen columns find their seats on sixteen meridian trusses at the inside panel points B_9. Since there are thirty-two meridians that must be made to share alike in carrying this platform loading to produce uniform deformation of the framework, a truss was designed to frame between meridians at the load-application points. The inside chord of the girt at panel points B_9 serves also as the top chord for the distributing truss.

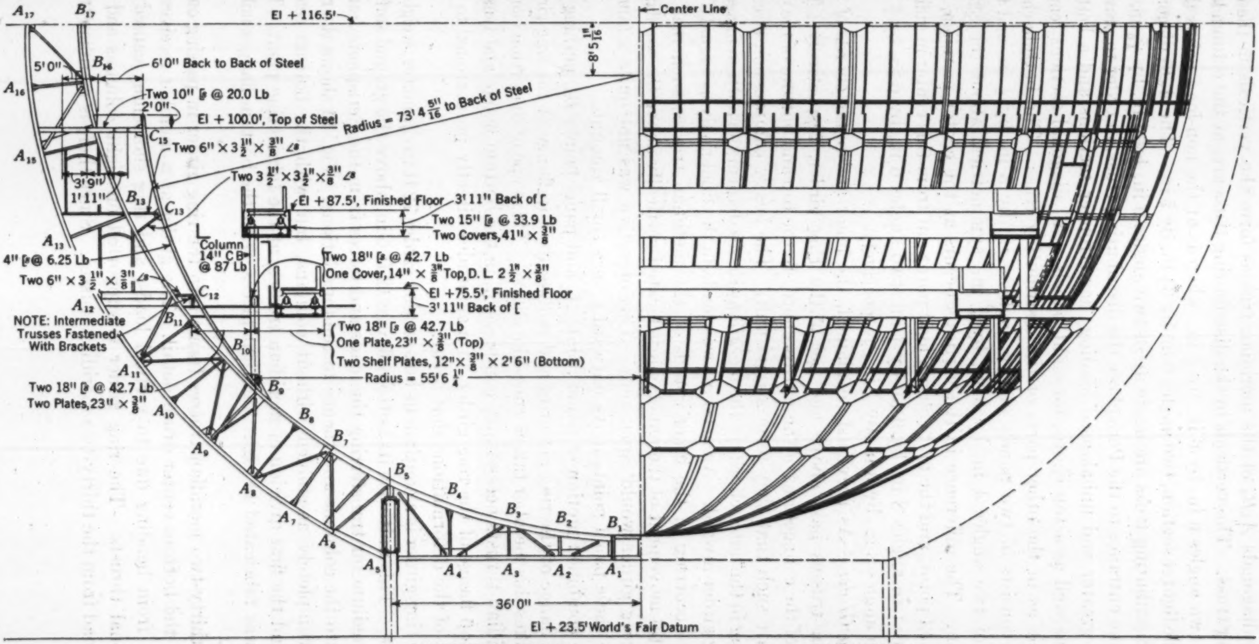

A17
B17
El + 116.5'
Center Line
8' 5 1/16"
5' 0"
B16
6' 0" Back to Back of Steel
A16
Two 10" [@ 20.0 Lb
2' 0"
El + 100.0', Top of Steel
Radius = 73' 4 5/16" to Back of Steel
C15
Two 6" × 3 1/2" × 3/8" ∠s
A15
3' 9"
1' 11"
B13
C13
Two 3 1/2" × 3 1/2" × 3/8" ∠s
El + 87.5', Finished Floor
3' 11" Back of [
Two 15" [@ 33.9 Lb
Two Covers, 41" × 3/8"
A13
4" [@ 6.25 Lb
Column 14" CB @ 87 Lb
Two 18" [@ 42.7 Lb
One Cover, 14" × 3/8" Top, D. L. 2 1/2" × 3/8"
Two 6" × 3 1/2" × 3/8" ∠s
C12
El +75.5', Finished Floor
A12
3' 11" Back of [
NOTE: Intermediate Trusses Fastened With Brackets
B11
Two 18" [@ 42.7 Lb
One Plate, 23" × 3/8" (Top)
Two Shelf Plates, 12" × 3/8" × 2' 6" (Bottom)
B10
A11
Radius = 55' 6 1/4"
Two 18" [@ 42.7 Lb
Two Plates, 23" × 3/8"
B9
A10
B8
B7
A9
B5
A8
B4
A7
B3
B2
B1
A6
A5
A4
A3
A2
A1
36' 0"
El + 23.5' World's Fair Datum
QUARTER SECTION
ELEVATION

Fig. 13.—Steel Framing, Lower Hemisphere

The diagonals A_8B_9 of the meridian trusses form the verticals in the distributing truss. The verticals in the distributing truss are, at the column load points, two angles 6 in. by 6 in. by $\frac{3}{8}$ in., whereas at the non-load points they are of the lighter section, two angles 6 in. by $3\frac{1}{2}$ in. by $\frac{3}{8}$ in. The A_8B_9 diagonals of the distributing truss are made up of two angles 6 in. by 4 in. by $\frac{7}{16}$ in.

At the entrance to the Perisphere the distributing truss, in order to provide for the greater and unbalanced loading, has stronger sections and a bottom chord, as well as a top chord, for eight panels. All diagonal sections remain the same as in the other parts of the distributing truss. The bottom-chord section consists of two panels of two angles 4 in. by 3 in. by $\frac{1}{2}$ in. and two panels of two angles 4 in. by 3 in. by $\frac{3}{8}$ in., symmetrical about the center meridian. The difference in the verticals occurs at the center meridian, the first panel point, and the first column load point out from this center meridian; these are two angles 8 in. by 6 in. by $\frac{3}{4}$ in., two angles 6 in. by 6 in. by $\frac{1}{2}$ in., and two angles 8 in. by 6 in. by $\frac{1}{2}$ in., respectively.

Ring Girder.—As previously explained, the inner and outer chords of the meridian trusses pass above and below the ring girder, respectively. The details of the connections of these truss chords to the top and bottom of the ring girder are such that they form a rigid unit. The heavy compression stresses occurring in the outer chords of the meridian trusses deform the bottom flange of the ring girder inward. An attempt was made in the final analysis to make the stresses occurring in the inner chords of the meridian trusses such that the horizontal movement of the panel points located vertically above the top flange of the ring girder would equal zero. This objective was not quite attained, because these panel points move outward a very small amount.

The combined motion of panel points A_5 and panel points B_5, pushing the bottom flange of the ring girder in and pulling the top flange of the ring girder out, makes this member take a cone-like shape. This type of distortion caused by meridian thrusts necessitates compression in the bottom flange and tension in the top flange of the ring girder, in magnitudes directly proportional to the amount of the deformations that occur.

The ring girder is a continuous box-section girder 72 ft in diameter, weighing 139 tons. This unit with its bottom flange 12 ft 6 in. above the ground surface is the medium for transferring the loads and forces from the Perisphere framework into the columns and thence to the foundations. Fig. 14 shows the ring girder completely assembled without columns, but with the bottom center drum and the first division of meridian trusses, at the fabricating plant. The girder was fabricated in eight identical pieces, with the bottom flanges milled to bear.

All thirty-two meridian trusses frame into the ring girder in passing on to butt at the bottom center drum detail. The girder as a member receives its stresses from bending due to vertical loads, and ring shortening caused by meridional thrusts. The ring girder is supported on eight columns and receives load from the thirty-two meridian trusses. Since there is meridian truss at each column, it follows that a section of girder between two adjacent columns is loaded at its midpoint and its two quarter points by the other meridian trusses.

In the final analysis of the 72-ft diameter girder for direct stress as a ring, panel points B_8 and A_8 were found to move out 0.015 in. and to move in 0.058 in., respectively.

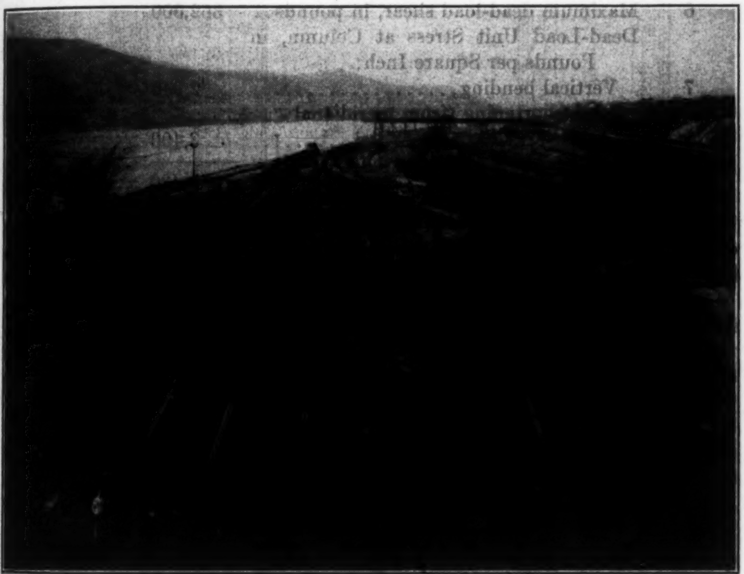


FIG. 14.—TEST ASSEMBLY OF RING GIRDER AND BOTTOM DRUM AT THE FABRICATING PLANT

The assumption was made that these deformations would create ring stresses in the circular girder. The flanges of the ring girder were then sectioned so as to produce the unit stresses accompanying the foregoing deformations—tension in the top flange and compression in the bottom flange. Since the ring-girder flanges are necessarily connected, one to the other, by the two $\frac{1}{16}$-in. webs, some participation in the ring action must be taken by these webs. After due consideration it was assumed that 45 in. of the webs played a part in the action, the maximum web participation being at the bottom flange and decreasing as a straight-line variation to zero at a point 45 in. up from the bottom flange.

The following data indicate the ring-girder sections used and the stresses from balanced loads:

Item	Description	
	Section:	
1	Two web plates	90 in. by $\frac{7}{16}$ in.
2	Four angles	8 in. by 6 in. by $\frac{3}{4}$ in.
3	One top cover plate	23 in. by $\frac{5}{8}$ in.
4	Two bottom cover plates	23 in. by $1\frac{3}{16}$ in.
5	Maximum dead-load moment, in pound-feet	2,133,000
6	Maximum dead-load shear, in pounds	362,000
	Dead-Load Unit Stress at Column, in Pounds per Square Inch:	
7	Vertical bending	2,940
8	Ring shortening (from meridional thrust)	3,490
9	Total	6,430

Bottom Drum.—After passing the ring girder the meridian trusses converge on a center drum detail. All bottom chords of the thirty-two meridian trusses bear with milled surfaces against the bottom plate of the center drum which is 10.5 ft in diameter and 2½ in. thick, as shown in Fig. 9. A 1-in. ring plate stiffened by plate and angle diaphragms, a top plate 11 ft in diameter and ¾ in. thick, and a top reinforcing plate ¾ in. thick make up the remainder of the drum. All meridian trusses are rigidly framed into the top, bottom, and ring plates of the drum detail.

Air Conditioning.—The complete air conditioning system provided for the Perisphere requires a fully equipped fan and compressor room. This room and equipment were placed within the ring girder and supported on the inner flanges of the meridian trusses. The reinforced concrete platform forming the floor of the room is placed on a series of circular, concentric, reinforced concrete beams resting on the panel points of the meridian flanges and in this manner distributing the superimposed load uniformly to the meridians. This load, placed within the ring girder, actually decreases somewhat the meridian stresses resulting from the action of the framework. The accompanying ductwork and details of the system are contained within the space provided between the outer and inner shells, and the necessary louvers and vents are provided near the top of the Perisphere.

Columns.—The make-up of the eight main columns, each weighing 9 tons, supporting the Perisphere is as shown in Fig. 15. Because of the framing of these columns into the ring girder and meridian trusses, they are fixed at the top. At the bases the columns are fixed, by anchor bolts 3 in. in diameter, about the radial axis 2–2; but in order to reduce temperature stresses the columns are hinged about the tangential axis, 1–1.

The hinging effect was accomplished, as will be noted in Fig. 15, by tapering the 5-in. base plate and providing the 8-in. by 8-in. by ½-in. breathing angles.

Wind Shear on Columns (See Fig. 16).—The distribution of the wind shear among the various columns was determined in the following manner: All columns are alike. At the bottom they are pin-ended about axis 1–1 and fixed-

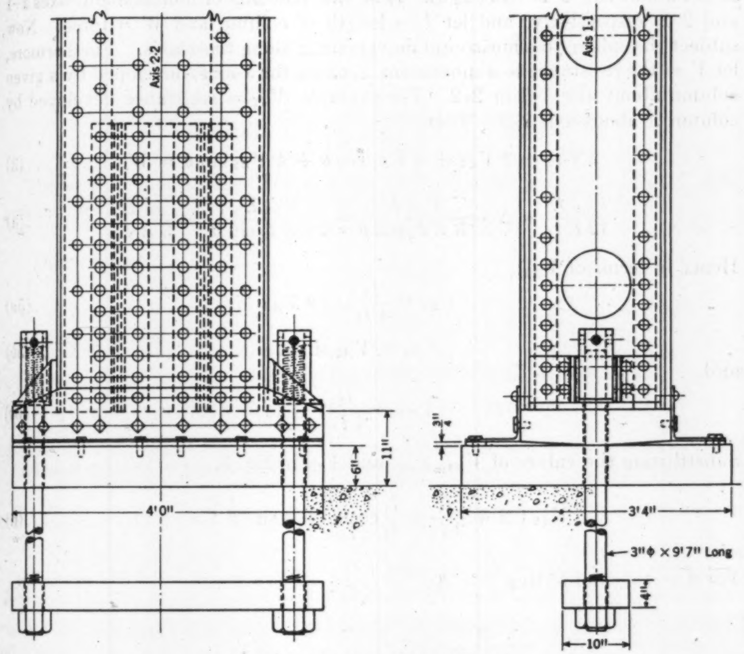

Axis 2-2
Axis 1-1
3/4"
11"
6"
4'0"
3'4"
3"φ × 9'7" Long
4"
10"

FIG. 15.—DETAIL OF COLUMN BASE

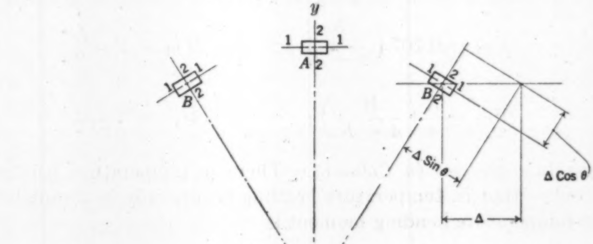

y
1
2
1
A
2
B
Δ Sin θ
Δ Cos θ
Δ
θ
W
x
C
Symmetrical About Center Line
x

FIG. 16

ended about axis 2–2. Let I_1 and I_2 be the moments of inertia about axes 1-1 and 2–2, respectively; and let L = length of column and W = force. Now subject the sphere to a horizontal movement Δ along the x-axis. Furthermore, let V = the resistance to a movement Δ, along the x-axis, developed by a given column about axis 1–1 or 2–2. For example, V_{A2} = resistance developed by column A about axis 2–2. Then:

$$2\,V_{A2} + 2\,V_{C1} + 4\,V_{B1}\cos\theta + 4\,V_{B2}\sin\theta = W \quad \text{(3)}$$

and

$$\frac{V_{A2}\,L^3}{12\,E\,I_2} = \Delta = \frac{V_{B1}\,L^3}{3\,E\,I_1\cos\theta} = \frac{V_{B2}\,L^3}{12\,E\,I_2\sin\theta} = \frac{V_{C1}\,L^3}{3\,E\,I_1} \quad \text{(4)}$$

Hence in terms of V_{A2}:

$$V_{B1} = \frac{I_1}{4\,I_2}\cos\theta\,V_{A2} \quad \text{(5a)}$$

$$V_{B2} = V_{A2}\sin\theta \quad \text{(5b)}$$

and

$$V_{C1} = \frac{I_1}{4\,I_2}\,V_{A2} \quad \text{(5c)}$$

Substituting the values of V_{B1}, V_{B2}, and V_{C1} in Eq. 3,

$$V_{A2}\left(2 + \frac{I_1}{2\,I_2} + \frac{I_1}{I_2}\cos^2\theta + 4\sin^2\theta\right) = W \quad \text{(6)}$$

For $\theta = 45°$, and letting $\frac{I_1}{I_2} = K$:

$$V_{A2} = \frac{W}{4 + K}; \qquad M_{A2} = V_{A2}\frac{L}{2}$$

$$V_{B1} = \frac{0.707}{4}K\left(\frac{W}{4 + K}\right); \qquad M_{B1} = V_{B1}\,L$$

$$V_{B2} = 0.707\left(\frac{W}{4 + K}\right); \qquad M_{B2} = V_{B2}\frac{L}{2}$$

and

$$V_{C1} = \frac{K}{4}\left(\frac{W}{4 + K}\right); \qquad M_{C1} = V_{C1}\,L$$

Temperature Stresses in Columns.—There is temperature bending about axis 1–1 only—that is, temperature bending occurs only in a radial direction. Then the temperature bending moment is

$$M_t = \frac{3\,E\,I_1\,\Delta_t}{L^2} \quad \text{(7)}$$

in which E = modulus of elasticity; I_1 = moment of inertia about axis 1–1; Δ_t = deflection due to temperature; and L = length of column. Substituting the values for a temperature change of 40° F: $M_t = 0.282\,I_1$ kip-in. $= \frac{0.282 \times 13{,}561}{12} = 319$ kip-ft.

Column Section.—The gross area, in square inches, is:

4 plates 30 by $\frac{11}{16}$	82.52
4 angles 8 by 4 by $\frac{3}{4}$	33.76
2 plates 20 by $\frac{5}{8}$	12.50
Gross area	128.78

The section moduli are: For axis 2–2, 917; and for axis 1–1, 1,164. The unit stresses are as follows:

$$\text{Direct stress}\ldots\ldots\ \frac{1{,}022{,}000}{128.78} = 8{,}000$$

$$\text{Bending stress}\ldots\ldots\ \begin{cases} \dfrac{7{,}872{,}000}{917} = 8{,}600 \\[2ex] \dfrac{1{,}915{,}000}{1{,}164} = 1{,}600 \end{cases}$$

Dead + Live + Snow + Wind, in pounds per square inch	18,200
Temperature	3,300
Dead + Live + Snow + Wind + Temperature	21,500

TRYLON

Because the wind forces governed the design of the Trylon, the columns as well as the anchorages are unusually heavy. The wisp-like appearance of the completed tower is belied by the fact that each tower-leg is anchored by means of a very heavy welded-riveted base and fourteen 2⅞-in. diameter bolts each 12 ft in length. The base and the anchor bolts of each column are embedded in a three-way reinforced mass of concrete. Special precautions were taken in designing each tower leg for the 855-ton uplift. As is noted in the data table, it was necessary to change a large portion of the covering on the Trylon in 1940. The new type of covering weighed only a little more than one third of the type it replaced. The decrease in the weight of the covering increased the maximum uplift load to 920 tons on a column.

First Column Section.—The gross area, in square inches, is:

1 cover plate 22 by 1⅛	24.75
2 angles 8 by 6 by 1	26.00
2 web plates 32 by 1½	96.00
2 angles 8 by 8 by 1	30.00
2 plates 24 by 1	48.00
1 cover plate 37 by 1⅛ (23 in. effective)	25.88
Gross area	250.63

$$\frac{l}{r} = 32; \qquad \text{allowable } f_c = 14{,}740 \text{ lb per sq in.}$$

The unit stresses, in pounds per square inch, are as follows:

$$\text{Dead load} \ldots\ldots\ldots \frac{1{,}100{,}000}{250.63} = 4{,}400$$

$$\text{Wind load} \ldots\ldots\ldots \frac{2{,}580{,}000}{250.63} = 10{,}300$$

$$\text{Dead + wind} \ldots\ldots\ldots\ldots\ldots\ldots \overline{14{,}700}$$

COVERINGS

The exterior covering of the Perisphere consists of 2-in. by 4-in. wood nailers, treated with a fire-retardant medium and screw-fastened to the vertical steel purlin system; two thicknesses of ½-in. gypsum wallboard, the first layer being damp-proofed with a full covering of asphalt emulsion, were applied vertically and nailed to the horizontal timber furring. Then three layers of a magnesite compound, reinforced with two layers of jute fabric, were applied to the entire surface of the Perisphere and painted white.

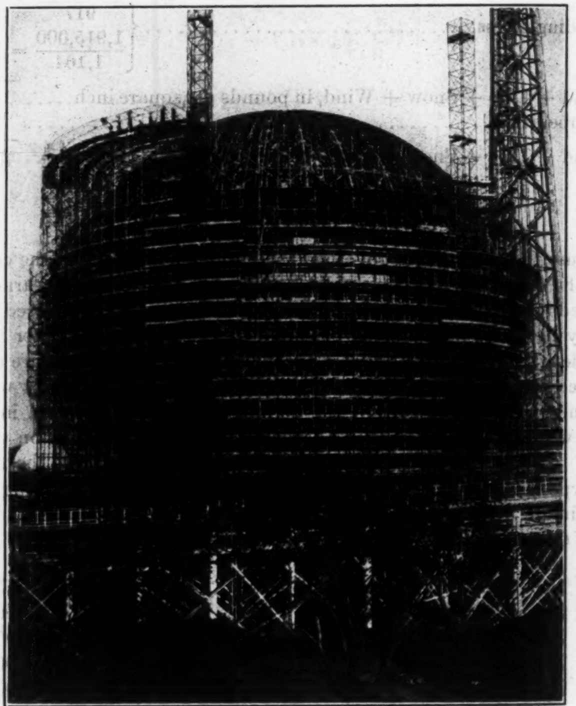


FIG. 17.—VIEW OF DEMOUNTABLE SCAFFOLD SYSTEM

The interior shell or lining of the Perisphere consists of an acoustical tile, upon which pictures are projected. The tile is nailed to 2-in. by 3-in. fireproofed horizontal furring, 27 in. on centers, attached to the inner steel purlins by means of heavy wire clips. On the area of tile extending from the offset in the inner shell below the equator, to a point approximately 30° from the zenith, the tile has a backing in alternate panels, in both directions, of a ⅜-in. plasterboard and a 1-in. sound-absorbing board.

The exterior covering applied to the steel purlins of the Trylon and Bridge was of the same type as that for the Perisphere except that there was only one thickness of gypsum wallboard. The outer coverings of the Perisphere, Trylon, and Bridge were applied by using an exterior, steel-pipe, demountable scaffolding system, as shown in Fig. 17.

The interior shell of the Perisphere above the level of the light-projector platform was constructed with the use of a revolving scaffold made of tied-arch trusses having their curvature fitted to the inner Perisphere surface. The three fan-like trusses were interlocked with X-bracing. The scaffold was suspended from the zenith of the sphere by a spider-and-pivot hanging support. At the light gallery (100-ft level), the scaffold has a set of rollers, one under each truss, and these rollers run on a circular I-beam track. By means of horizontal platforms projecting out at various levels from the arched scaffold, workmen were able to apply a portion of a lune of inner surfacing and then move the scaffold around on its track into a new position, lock the rollers, and apply the surfacing to another lune. This procedure was repeated until the upper inner shell was completed.

The tied-arch scaffold framework, being bolted together, was easily dismantled upon completion of the surface, and, if necessary, can be put up again readily for maintenance purposes.

FABRICATION

Both the 1,000 tons of Trylon steel and the 2,000 tons of Perisphere steel were fabricated and erected by the American Bridge Company.

It was necessary to curve all members of the Perisphere, except the columns, webs of trusses, and interior framing, in order to fit the spherical surfaces. As a result of bending these members to arcs of great circles, as previously mentioned, the amount of work was held down to a minimum. More than one quarter of a million rivets, of which 100,000 were used during erection, were driven in order to fabricate and erect the 6,600 individual pieces of the Perisphere, varying in weight from a few pounds to 18 tons.

The curved members were bent cold by the use of jigs and bending machines. All curved trusses of the Perisphere were laid out and fabricated in specially designed jigs. The field connections in the meridian trusses were reamed assembled so as to minimize the inaccuracies and assure the best obtainable construction.

The bottom center drum and the ring girder, constituting the "nucleus" from which the Perisphere was constructed, are shown assembled at the fabricator's plant in Fig. 14. The ring girder was fabricated in eight sections and fitted to form a perfect circle 72 ft in diameter.

ERECTION

Trylon.—The 615-ft Trylon was erected by means of a 70-ft basket boom. The column steel came in sections varying from 19 ft to 33 ft in length, and when the basket boom had surrounded itself with tower framework, it was lifted and the guys fastened in higher positions provided for in the column details. By this method the Trylon, including the top steel-plate section, was completely erected.

Perisphere.—A caterpillar-tractor crane having a 100-ft boom and a 12-ft jib was used to erect the Perisphere steel to within a short distance of the equator. Before reaching this stage of construction, however, the tractor crane set up an erection tower 30 ft square and 90 ft high on the inside of the Perisphere. The tower rested on, and was fastened to, the inner chords of the meridian trusses within the ring girder. Two 12-ton stiff-leg derricks set on diagonally opposite corners of the tower erected the remainder of the Perisphere with their 97-ft booms.

In general, the procedure of erecting the Perisphere was that of first placing one section of each of the meridians and then inserting the corresponding units of the girt trusses. Then the next sections of meridian and girt trusses were bolted up, and so the sequence was repeated until completion. The riveting followed the erection very closely so as to have all connections riveted before very much load came on the framework. No camber was provided in any part of the structure; consequently, the completed Perisphere has its natural dead-load deformation.

Due to the process of erection (placing a complete section of corresponding meridians and girts before proceeding to the next section) all members were erected in practically an unstressed condition. At some stages of the erection, the unsymmetrical temperature effects on the steelwork were enough to create gaps of 1½ in. on closing units of girt trusses. This opening was taken care of by adjustment back into all of the thirty-two units of a girt truss so that in following such method no reaming of any connections was necessary in the entire erection of all the meridians and girts.

It is worthy of note that, although these structures were unusual in both type and details, no unforeseen difficulties arose, during either the fabrication or the erection, that involved delays or required revisions.

ACKNOWLEDGMENTS

The Theme Center was built by the New York World's Fair 1939, Inc., under the direction of the Department of Construction, with John P. Hogan, President, Am. Soc. C. E., chief engineer, and L. B. Roberts, M. Am. Soc. C. E., assistant chief engineer.

The design and plans of the steelwork for the Perisphere, Trylon, and Bridge were prepared by Waddell and Hardesty under the general direction of Mr. Hardesty, those for the Perisphere being supervised by Mr. Hedefine, and those for the Trylon and Bridge by Francis De Schauensee, M. Am. Soc. C. E. The foundations were designed by Moran, Proctor and Freeman, and the architects were Harrison and Fouilhoux. The original manuscript, complete with bibliography, has been placed on file for reference at The Engineering Societies Library, 29 West 39th Street, New York, N. Y.

DISCUSSION

Louis Balog,[5] Esq. (by letter).—The five Perisphere designs mentioned by the authors were submitted to the Construction Department of the Fair for consideration on April 12, 1937. One of these, designated as Design No. 1, was adopted for construction. The major part of the paper presents a detailed account of this design.

Design No. 1 consists of a steel framework and a separate outer covering. The spherical steel framework is composed of girt and meridian trusses of a depth of $\frac{1}{16}$ to $\frac{1}{8}$ of the outside radius. Assuming pin connections at the intersection of the meridians and girts the stresses in the framework, due to symmetrical loads, can be determined from equilibrium requirements. If the connections are rigid, the elastic deformation of the girts results in meridian moments. The magnitude of these moments is primarily a function of the moment of inertia of the meridians. The deeper the meridians the larger the moments will be due to elastic deformation of the girts. In case of n girts the meridian moments can be obtained by the solution of $n - 1$ elastic equations expressing the equality of the elastic displacements of girts and meridians at their intersections. The operations involved are not too formidable; but they still are not justified by the significance of these moments as compared to those that result from the curvature of the meridians between two intersection points. For unsymmetrical loadings, like those due to wind, the number of elastic equations increases so far beyond any reasonable limit that a rigorous investigation becomes impracticable.

To simplify the analysis the authors divide the sphere at the equator. They assume the upper hemisphere to be a statically determinate structure and obtain the stresses from equilibrium requirements. In the lower hemisphere the meridians were assumed to be cantilevers resting on a series of elastic supports, represented by the girts. These assumptions result in a sufficiently close approximation of the stresses in the framework due to axially symmetrical loadings, such as the dead and snow loads. The wind-pressure distribution, as given from tests, is not symmetrical to any plane containing the vertical axis of the sphere. It is not adaptable to mathematical expression that is suitable for the derivation of closed formulas for the wind stresses. The authors' analysis resulted in safe values for the stresses produced by the assumed wind-pressure distribution as comparative calculations made by the writer show. The diagonal bracing provided, and the large moment-carrying capacity of the deep meridian trusses, make this structure well fitted to carry unsymmetrical wind loads.

Although rigorous investigations indicated that the approximate methods used by the authors are suitable for design purposes, it is regrettable that no stress measurements were made on the structure as built. Such measurements would have given invaluable hints for correct simplified analysis, the only kind that is of genuine value in the design of this type of complex structure. This holds especially for the wind stresses. Extensive laboratory investiga-

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[5] Engr. with Leon S. Moisseiff, Cons. Engr., New York, N. Y.

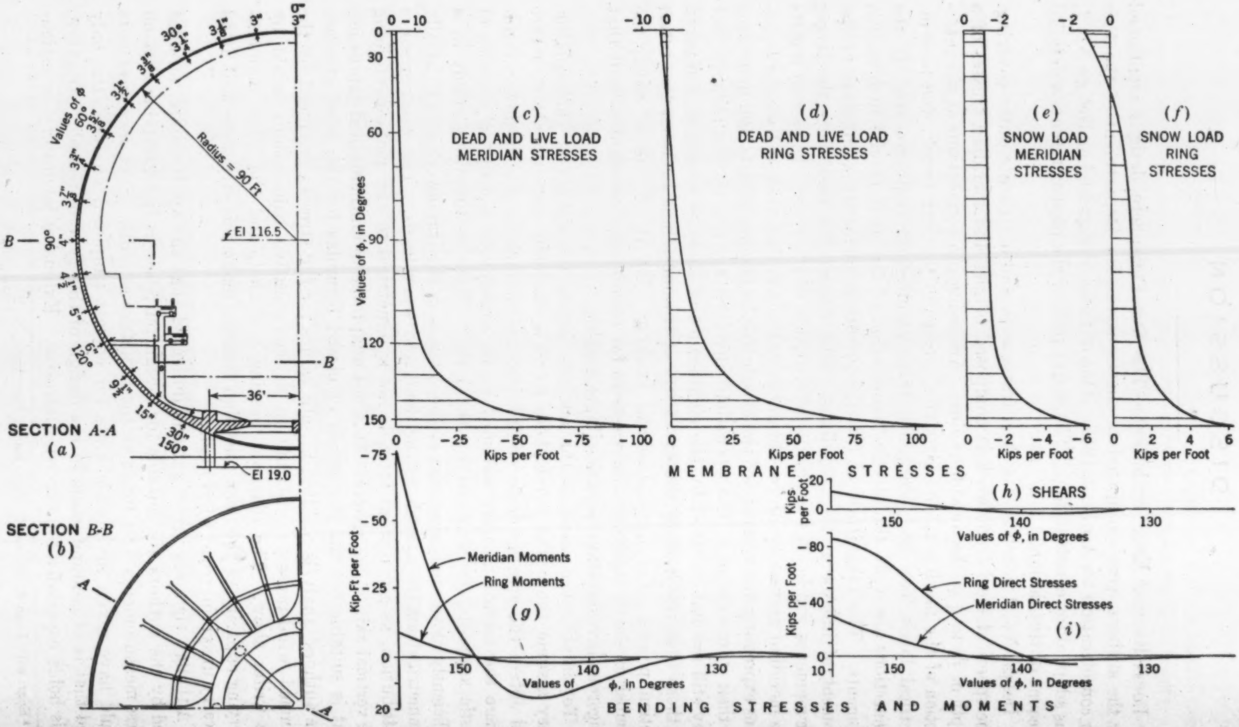

FIG. 18.—SECTIONS AND PRINCIPAL STRESSES OF CONCRETE PERISPHERE

tions were made to determine the magnitude and distribution of the wind loads. The stresses produced by these loads, however, were not ascertained by an experimental method. The authors' skilful estimate of the stresses in the framework due to wind would be of considerably greater value if measurements had shown how closely the actual conditions were approximated.

The problem in the design of the Perisphere was to create a structure which carries specified loads safely and has a smooth spherical surface. The latter requirement necessitates the use of a complicated outside covering on a steel framework. By the use of reinforced concrete, such covering is eliminated and the carrying structure can be formed readily to the desired shape and surface.

Design No. 5, a reinforced concrete shell structure, was proposed and designed by the writer. It differs, not only in its material but also in structural conception, from the other four designs, which were ribbed steel structures. The principal characteristics, design data, and quantities of this design are presented in the following:

The great carrying capacity of shells in general and that of shells of double curvature, like the sphere, in particular, would permit the use of very thin sections. In the upper two thirds of this spherical shell the thickness was determined by construction considerations rather than by the stresses, the safety factor against buckling, or durability requirements. The construction of a sufficiently precise formwork and the assurance of careful workmanship in placing the reinforcing steel and concrete constituted the difficulties that influenced the selection of the minimum thickness. These considerations resulted in the adoption of greater thickness for the upper part of the shell than that of existing permanent structures with similar radius of curvature.

Fig. 18 indicates the principal dimensions and stresses of the reinforced concrete sphere. The outside diameter is 180 ft. It is supported on eight round columns, 4 ft in diameter, arranged similarly to the steel design.

The thickness of the shell increases slowly from 3 in. $\left(\frac{1}{360}\right.$ of the outside radius$\left.\right)$ at the top where $\phi = 0°$, to 4 in. $\left(\frac{1}{270}\right.$ of the outside radius$\left.\right)$ at the equator where $\phi = 90°$. The increase in thickness below the equator is more rapid. At $\phi = 140°$ where the live load on the moving platforms, their weight, and that of their reinforced concrete supports are transmitted, the shell thickness is 15 in.—$\frac{1}{74}$ of the outside radius. Below this point the shell is stiffened by meridional ribs, its thickness rapidly increases and at the face of the support ring reaches 36 in.—that is, $\frac{1}{30}$ of the outside radius. The ratios of thickness to radius indicate that the membrane theory of shells, which assumes small thicknesses and uniform distribution of the strains in the cross section of the shell, is applicable to this structure except in the vicinity of the support ring. This assumption of uniform strains results in the disappearance of the bending and torsion moments perpendicular to the center surface of the shell and permits the calculation of the direct stresses, caused by axially symmetrical loadings, from equilibrium requirements.

The dead load and similarly distributed loads produce direct stresses in the direction of the meridians and parallel circles, causing small deformations of the shell. Rigorous investigation of spherical shells[6] indicate an upper limit of $\frac{\delta^2}{2\,a}$ for the magnitude of these deformations. For a shell thickness of $\delta = 4$ in. and a radius of $a = 90$ ft the largest eccentricity is thus 0.0074 in. The bending moments introduced by such deformations are negligibly small. The moments produced by the support conditions, however, are considerable. The membrane stress condition at the support ring is thoroughly disturbed resulting in large bending stresses in the direction of the meridians. It was specified that the outside surface of the shell should be a perfect sphere. For this reason the thickness of the shell increases inward and the middle surface, which bisects the thickness of the shell, deviates from a true spherical surface. The radii of curvature of this surface do not differ significantly from that of a sphere, except in the vicinity of the support ring where the change of its curvature is considerable and unfavorable. At points where the curvature of the middle surface changes suddenly, the change of the ring stresses is abrupt, and bending stresses are created in the direction of the meridians. By varying the shell thickness so that the middle surface is represented by a practically continuous function the abrupt changes in the ring stresses and the introduction of moments can be eliminated.

The dead and live loads were assumed to be symmetrical about the vertical axis of the sphere. The symmetry of the dead load and the smallness of the live load (about $\frac{1}{30}$ of the dead load) justified this assumption. The snow load was assumed to be 25 lb per sq ft at the top of the sphere, diminishing as the cosine of ϕ to zero at the equator. This assumption represents a total snow load about 60% larger than that used by the authors for the steel design, although that assumed by the authors was ample for any condition that would have occurred. The effect of this load, which is about $\frac{1}{20}$ of the dead load, is small in the lower part of the shell. The computation of the wind stresses due to the given pressure distribution could be only approximate. These stresses, as compared to the dead-load stresses, have significant values only at the upper part of the sphere where the shell thickness, determined by construction considerations, assures small unit stresses. The advantageous relation between the dead-load and wind-load stresses assures that the shell deformations produced by the unsymmetrical wind loads are small enough to make the bending moments introduced by these deformations negligible.

The maximum membrane stresses in the concrete produced by the combined dead, live, snow, and wind loads do not exceed 188 lb per sq in. tension and 224 lb per sq in. compression.

At $\phi = 155°$ the shell is joined to the support ring which rests on eight columns. This ring is compressed radially by the horizontal component of the meridian stresses. The abrupt change of the ring stresses at the support ring produces bending and shearing stresses perpendicular to the middle surface of the shell. Temperature difference between the exposed shell and the inclosed support ring also has the same effect. Fig. 18(*g*), (*h*), and (*i*), shows the effect of the support ring on the shell. These stresses are due to dead, live, snow, and wind loads and 20° F temperature difference between the shell and the support ring. They were computed with consideration of the variation of the thickness of the shell.[7] Both vertical and horizontal components of the meridian stresses create bending stresses that are carried by the support ring in cooperation with the shell. The column reactions also effect additional stresses in the shell. The significant values of all of these stresses are localized in the vicinity of the support ring. They were determined by the application of theories developed primarily by Fr. Dischinger[8] and W. Flügge.[9] The maximum concrete unit stress produced by the combined effect of all loads and 20° F temperature difference between the shell and support ring is 574 lb per sq in. compression. To aid the distribution of the concentrated loads and stiffen the shell near the support, meridional ribs were arranged below $\phi = 140°$. Between the columns a shell 2.5 in. thick was suspended from the support ring. It has no statical purpose; it only completes the sphere.

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[6] "Spannungen in Kugelschalen," by H. Reissner, *Müller Breslau Festschrift*, Kröner, Leipzig, 1912, p. 181.

The quantities of the structure are as follows:

Item	Concrete, in cu yd	Reinforcing steel, in lb
Shell	1,860	650,000
Ribs, framing and support ring	1,035	209,000
Suspended shell	29	6,000
Columns	56	25,000
Total	2,980	890,000

The writer believes that the costs given by the authors is high for Design No. 5. The construction of the lower part, up to $\phi = 120°$, where 70% of the quantities are located, does not differ from common reinforced concrete jobs, except that the falsework supporting the forms of this part should be kept in place until the concrete of the entire structure hardens. The building of the upper part does not involve considerably greater difficulties than the construction of the numerous large shells which have overcome, successfully, the competition of framed steel structures. The writer is inclined to believe that bids based on the working drawings of the concrete design would have more than justified the cost of extensive model testing.

The authors showed ingenuity in the analysis and great structural skill in the layout and design of the steel structure. This is demonstrated by the fact that this unusual structure could be both fabricated and erected without difficulties.

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[7] "Über die Festigkeit Achsensymmetrischer Schalen," by J. Geckeler, *Forschungsarbeiten*, Heft 276, 1926, p. 19.

[8] "Die Rotationsschalen mit unsymmetrischer Form und Belastung," by Fr. Dischinger, *Der Bauingenieur*, Vol. 16, 1935, p. 393.

[9] "Die Statik und Dynamik der Schalen," by W. Flügge, Springer, Berlin, 1934, p. 43.

SHORTRIDGE HARDESTY,[10] M. AM. SOC. C. E., AND ALFRED HEDEFINE,[11] ASSOC. M. AM. SOC. C. E. (by letter).—The writers wish to express their appreciation to Mr. Balog for the extension and development of Design No. 5 in his discussion of their paper. Mr. Balog made the design of the reinforced concrete shell structure; and his design was then checked and included as one of the five designs submitted for consideration to the Construction Department of the Fair.

After studying the information obtained from a number of sources as to the advisability of building the concrete type and its probable cost, the figure that appears in the paper was determined. The writers do not believe this estimate of cost for the concrete design was unduly high. The adopted steel design was built at rather high unit prices because of the unprecedented problems involved; but the same factors would have operated in the case of the concrete design.

In the preliminary design studies of the Perisphere, a set of simultaneous equations was written expressing the elastic displacements of meridians and girts at their intersections. This process was discarded early, because it proved impracticable and too complex. The structure, as stated in the paper, was extremely sensitive to applied loads. Consequently, any load changes (and there were many), even though relatively small, involved large variations in the coefficients of the simultaneous equations and made their solutions laborious and time consuming.

The writers agree with Mr. Balog that it is regrettable that no stress measurements were made on the structure as built. On several occasions attempts were made to arouse sufficient interest to secure money for this feature, but they were unsuccessful.

The structures, comprising 3,200 tons of steel and four acres of covering, displays, and equipment, were completely demolished in the spring of 1941. In the main, the procedure followed in demolition was the reverse of that used during construction for both the Trylon and the Perisphere.

It was hoped that some discussion would be presented relative to the possibility of designing a sphere of this size by making a steel shell self-supporting without the use of any framing, which would have been a very complicated problem since there were really two spheres, one inside the other. Such a type of construction was suggested in the early stages of the design, but was discarded because of the practical problems of fabrication and construction involved.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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Paper No. 2125

AXIOMS IN ROADWAY SOIL MECHANICS

By Henry C. Porter,[1] M. Am. Soc. C. E.

With Discussion by Messrs. Victor J. Brown, and A. A. Eremin.

Synopsis

In the proper design and construction of roadways, the detailed study of soil mechanics in all of its phases is complicated by the many items that must be understood and taken into consideration. Many different types of soils must be investigated; their characteristics must be predetermined; and they must be properly used in the roadway. The mechanics of the different types of soils as placed in the roadway are different. Practical methods of designing and constructing different parts of the roadway are not the same. Topographical and climatic conditions in different localities vary and also must be taken into account. There is no panacea, and rules of thumb cannot be established which will fit all conditions. Those who design, construct, and maintain the highways must know the fundamentals of soil mechanics, and use their knowledge to fit each individual situation properly. On the other hand, there are many axiomatic facts, thirteen of which are presented in this paper for detailed examination and comment. These axioms pertain mostly to volumetric changes in soils due to fluctuations in moisture content caused directly by alternate seasons of slow rains and droughts.

Definitions

During the beginning of the study of soil mechanics, one's first impression is that there are so many factors involved that they never can be correlated sufficiently for practical usage. This is an erroneous impression. In the first place, topics which are not fully comprehended appear difficult and unreliable, but when understood thoroughly and put into usage, they are simple, and other phases of the subject then come to light from time to time. In the second place, many laws of nature involving soil mechanics which have been known by highway engineers for many years have been disregarded in the building of roadway "fills and embankments."

Note.—Published in February, 1940, *Proceedings*.

[1] Engr., Soils and Research, State Highway Dept., Austin, Tex.

Because different principles and methods are involved in the proper design and construction of various parts of the roadway, and in order that these parts may be referred to, conveniently and definitely, the roadway is divided vertically into three general parts, as illustrated in Fig. 1, and defined as follows:

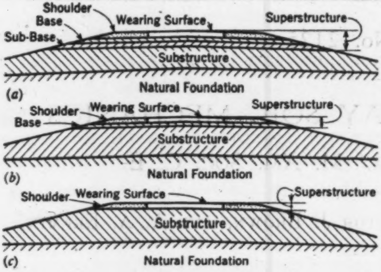

Fig. 1.—Vertical Subdivision of the Roadway

A. The Natural Foundation: The non-manipulated natural structure upon which the "substructure," the subbase, the base, or the wearing surface, placed directly thereon, depend for their support;

B. The Substructure: The man-made part of the roadway lying between the top of the natural foundation and the bottom of the "superstructure," where the soil types therein are predetermined and the structure is designed in accordance with known mechanics and chemistry of the soils (fill and embankment are terms used where the soils were not predetermined and the structure was not designed in accordance with present knowledge of soil mechanics and chemistry, as described under the heading "Substructure"); and

C. The Superstructure: (1) The subbase, base, and wearing surface, or (2) the base and wearing surface, or (3) the wearing surface only, where no subbase or base are used (when the superstructure is built by stages, the first wearing surface eventually may become the subbase, or the base of the completed superstructure).

Axioms

Certain phases of soil mechanics and their effects on overlying riding surfaces are now so well known that they may be called axioms in roadway building. These axioms should be used immediately and constantly in the preparation of all natural foundations and in the design and construction of all substructures until their usage becomes a habit. Other findings concerning this subject then will be made from time to time and put into usage in such a way that the cost of construction and maintenance of the roadways will be reduced continually and the efficiency increased. These fundamentals, or axioms, where properly applied, will not only return the greatest and quickest dividends in roadway construction and maintenance, but also will make a good foundation upon which a practical knowledge of soil mechanics may be built.

Some of the most important known factors of soil mechanics involved in the proper design, construction, and maintenance of roadways are named and discussed as follows:

> Axiom 1. To a large extent, the behavior of the superstructure depends upon the nature and amounts of movements in the natural foundation and in the substructure.

Fig. 2 shows the smooth riding surfaces retained by the pavement where the soil substructure is non-expansive sand (profiles Nos. 1 and 2), and rough riding surfaces that developed shortly after completion of pavement on the same project, where the soil substructure is expansive clay soil (profiles Nos. 3 and 4).

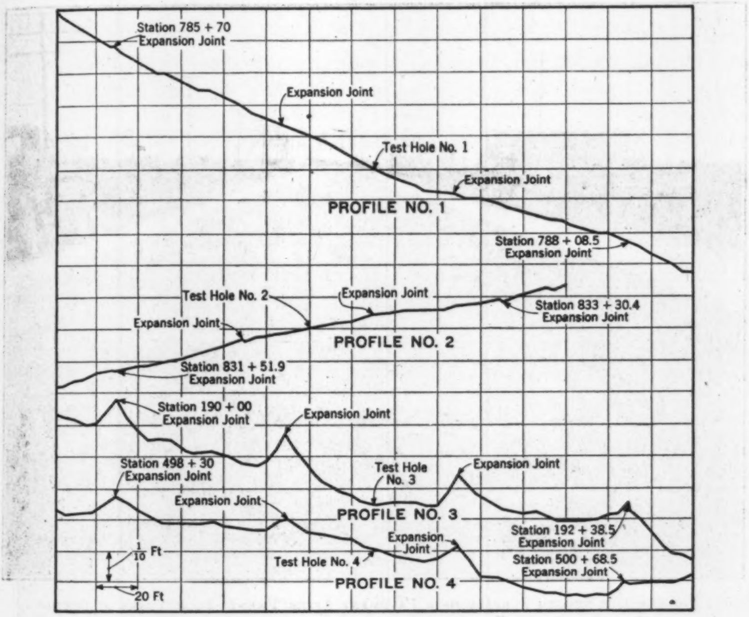

FIG. 2.—PROFILE AT CENTER LINE OF CONCRETE PAVEMENT

The pavement rose at the expansion joints where water leaked and wet the underlying clay soil.

Axiom 2. Many cohesive soils will support ordinary traffic loads without moving appreciably when they are relatively dry and are compacted in the natural foundation and substructure.

Axiom 3. Generally, the factor that causes density and volumetric changes or movements in the natural foundation and in the substructure is the flunctuation of moisture or water content in the soils.

A section of concrete highway, in Navarro County, Texas,[3] 1,500 ft long, was constructed in the winter and spring of 1931. It had a clay soil foundation of high volumetric change and was laid immediately after a long period of slow rains, when the soil was probably expanded to its maximum. Observations during the following summer indicated that there were more up and down movements along the edges where the moisture-content fluctuations were

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[3] *Proceedings*, International Conference on Soil Mechanics and Foundation Engineering, Cambridge, Mass., Vol. II, 1936, p. 256; also Fig. 1, p. 257.

greater than along the center of the pavement. The fluctuations along the edges were 0.4 ft, whereas the movements along the center line were only 0.2 ft. The clay soil under the edges was exposed to seasonal wetting and drying more than the soil under the center of the pavement.

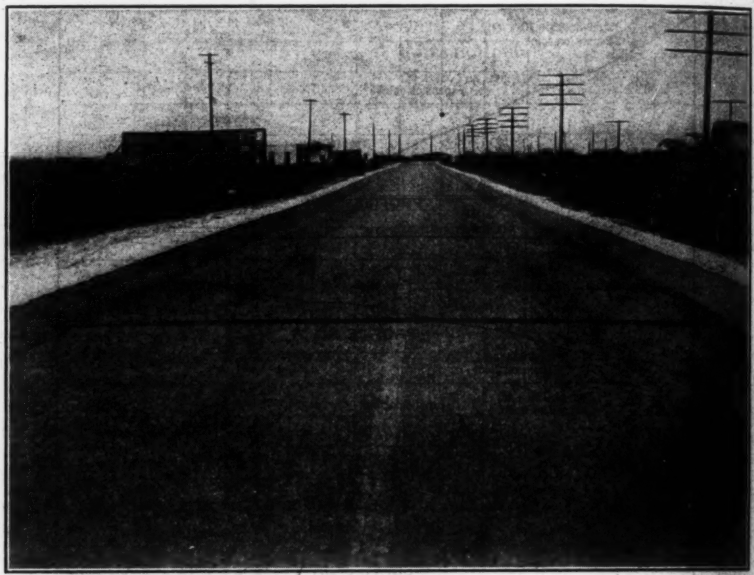

Fig. 3.—The Smooth Surface of a Pavement After Twenty-Four Years of Service

> Axiom 4. Soils of different types behave differently when subjected to moisture-content fluctuations. Coarse-grained material of inappreciable volumetric change and plasticity, with moisture-content fluctuations, such as sand, does not move appreciably, whereas fine-grained, highly expansive and plastic clays do move appreciably.

This axiom is illustrated by Figs. 3 and 4, showing the riding surface of a pavement after 24 years of service. This pavement carries heavy seaport traffic and, for more than a week after the 1919 tidal wave along the Texas seacoast, it was submerged beneath several feet of water. This highway has retained its smooth riding surface continuously with small maintenance costs.

To study the reasons for this long life, a section of the pavement was cut away. It was found that the underlying soil was composed of sand and shells which do not move with moisture-content fluctuations. At the place shown in Fig. 4, the pavement was only 2.5 in. thick.

> Axiom 5. A sloping smooth surface of compacted, fine-grained, clay soil tends to shed rain water, but when water is retained in contact with clay soil, continuously, the water eventually will permeate the clay sufficiently to cause it to expand in volume and become plastic.

Axiom 6. When rain water first falls on coarse-grained, pervious soil, such as sand, the line of least resistance to the movement of the water is into the large spaces or voids in the sand. Where sand rests on a sloping clay soil, the water will percolate through the sand until it arrives at the top of the impervious clay and the tendency then is for the water to move laterally through the sand along the top of the clay.

Water is furnished to most wells by movement of the water through strata of sand and gravel—sometimes through seams and crevices in shale and rock, but never in appreciable quantities through fine-grained clay soil. Generally, clay is used for making jugs, and sand is used for filters. Where the sand layer rests on and is flanked by clay soil on all sides, rain water often is impounded in the large voids of the sand on the clay soil and eventually permeates the clay soil sufficiently to cause it to expand in volume and become plastic.

Four views of a model test to study rainfall saturation are shown in Fig. 5. In Fig. 5(*a*) the rain is simulated by the manipulation of a flask and a number of wicks. It is shown running along the surface of the sloping clay soil. In this instance the sand (which extends to the intersections of the side slopes) does not impound the water on the under-lying clay soil; and, consequently, the clay soil has not been wet appreciably in 45 min. The reverse effect is shown in Fig. 5(*b*) in which the sand was entrenched in the clay soil. In that case the water had wet the underlying clay appreciably in 45 min.

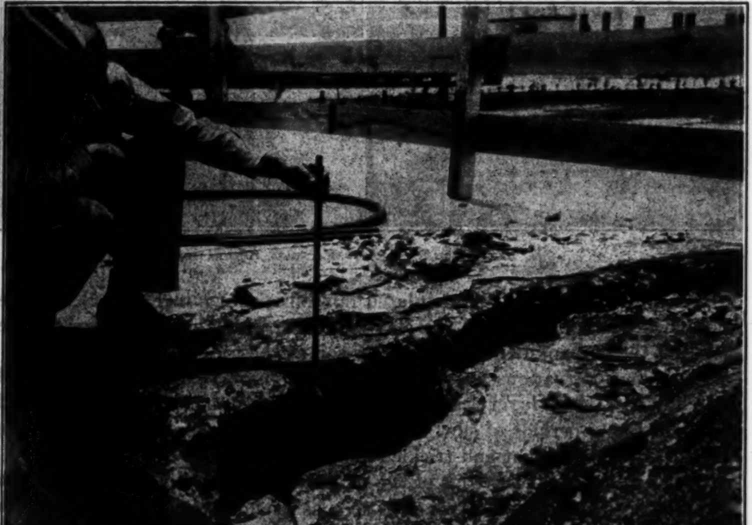

FIG. 4.—SECTION OF PAVEMENT IN FIG. 3 UNCOVERED TO STUDY REASONS FOR ITS LONG LIFE

After testing the foregoing two models for 19.5 hr, the results were as shown in Figs. 5(*c*) and 5(*d*). In that time, Fig. 5(*a*) became saturated to the extent shown in Fig. 5(*c*). In this case shrinkage cracks in the clay soil permitted the water to penetrate deeply into the soil substructure at those places.

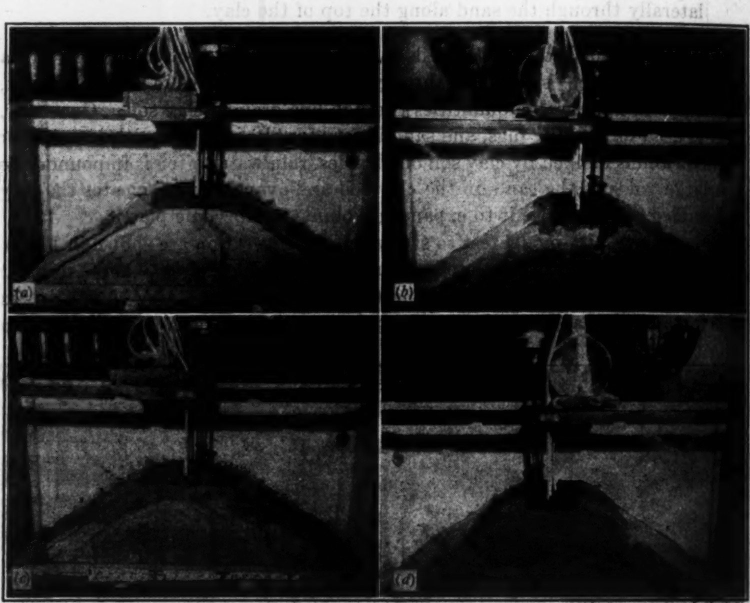

FIG. 5.—MODEL STUDIES OF RAINFALL SATURATION

Fig. 5(*b*) after 19.5 hr of simulated rainfall is shown in Fig. 5(*d*). All the water has been absorbed by the clay soil. No water has run down the side slopes of the embankment, and the clay soil at the toes is still dry.

In 1935, observations were made on a section of Texas highway that had been laid on a 12-in. layer of sand in a trenched clay subgrade.[8] It was found that, after long periods of slow rain, water rose through contraction joints to the top of the pavement and trickled along the top.

Holes were dug at the edge of the pavement in two adjacent sections. In Fig. 6(*a*) the shoulder was clay, and as soon as the hole was dug the water began to flow from the sand beneath the pavement. After one hour water had accumulated in the hole to a depth of 3.5 in.

At the adjacent section, conditions were the same as in Fig. 6(*a*) except that the 12-in. layer of sand extended as a blanket entirely across the crown to intersections with the side slopes of the roadway. Water has never risen to the top of the pavement at this section, and the test hole (see Fig. 6(*b*)) remained entirely free of water.

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[8] "Roadbed Design for Self-Drainage," by Henry C. Porter, *Engineering News-Record*, June 16, 1938, Fig. 1.

The centrifuge moisture equivalent (CME) test gives information as to the relative porosities or draining qualifications of different types of soils, and is one of the most important of the soil-constant tests. The determination of the relative draining characteristics of the different soils used is necessary for the proper design, construction, and maintenance of modern roadways, not only for the superstructure, but also for the substructure and the natural foundation.

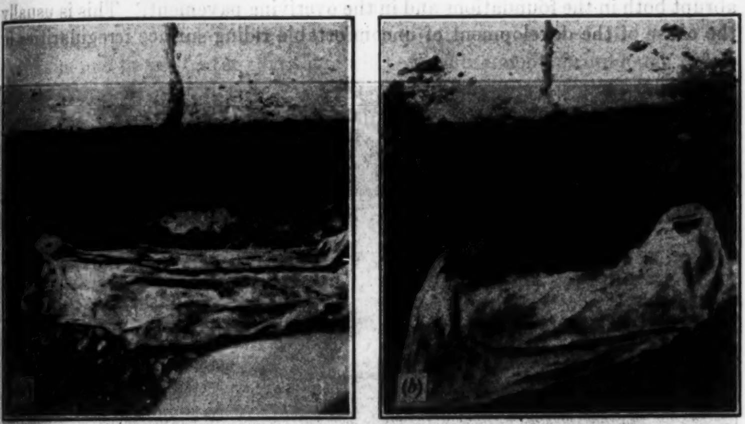

Fig. 6.—Comparative Test Sections at Edge of Pavement

Axiom 7. When the moisture content of expansive, fine-grained, clay soil increases above the shrinkage limit of the soil, the soil expands in volume, and the expanding soil will exert great lifting force.

It is known that some expanding clays will raise a weight of 50 lb per sq in. appreciably.

Axiom 8. When the soil type beneath the superstructure is uniform, when the soil is compacted to a uniform density with a uniform moisture content at the time the superstructure is laid, and when the subsequent moisture-content fluctuations in the soil are uniform throughout the entire length and width of the superstructure, then the superstructure will move up and down uniformly and will retain its smooth riding surface.

Axiom 9. The conditions enumerated in Axiom 8 can be obtained only by proper preliminary investigations, design, supervision of construction, and maintenance.

Often, merely treating the so-called "subgrade" is not sufficient to overcome the subsequent ill effects of all mechanical defects in fills and embankments and in natural foundations. Adequate preliminary investigations, design, and supervision of construction are necessary primarily because of the following phenomena:

(*a*) Much of the natural structure was formed by sedimentation and is in layers of materials of different types—perhaps a layer of clay, a layer of caliche, and a layer of sand, gravel, or rock. For example, in Fig. 7, the top layer is sandy loam, the second is clay, the third is sand, the fourth is clay, and the fifth stratum is gravel with a calcareous binder. Hence, when a pavement is laid to a grade on natural undisturbed structure (on an eroded hill-slope or in a hill-cut, for example), the foundation of the pavement may be composed of materials that will move by different amounts when subjected to moisture-content fluctuations. The changes in the amounts of movements very likely will be abrupt both in the foundations and in the overlying pavement. This is usually the cause of the development of uncomfortable riding surface irregularities in cuts, on hill-slopes, and at the junction of cuts and fills. During construction, irregular natural soil formations in a cut should be excavated to below bottom-of-superstructure grade line and backfilled in such a manner that the undisturbed natural structure will not cause the development of irregularities in the superstructure.

FIG. 7.—STRATIFIED NATURAL FORMATIONS OF MATERIALS OF DIFFERENT TYPES

(*b*) Materials of heterogeneously deposited natural formations often must be used for constructing substructures. If these materials are placed in fills and embankments (without selection), as generally has been done in the past, the change from highly expansive to non-expansive soil in the fills will be abrupt in many instances. Furthermore, pockets of coarse-grained soil often will be formed which, eventually, will impound water in the fills, as illustrated in Figs. 5 and 6. To overcome these mechanical defects the nature of the soils must be predetermined, by means of the prepared soil profile, and the substructure properly designed—even as superstructures are designed now. Where soils of different types must be used, the most impervious and highly expansive soil generally should be placed at the bottom of the substructure where the soil will be protected most from rain water moisture-content fluctuations, and so on, until the most pervious and non-expansive soil is placed at the top of the substructure. The top of each layer of soil should slope continuously from the center line to intersections with the roadway side slopes, for internal drainage. Where a layer of coarse-grained, pervious material is placed on impervious soil, particular care must be taken that the coarse-grained material is not flanked by finer-grained and less pervious soil.[4] When it becomes necessary to make a longitudinal change from one type of soil to another in any particular layer, the change should be gradual so that there will be no subsequent abrupt longitudinal difference in movements.

> Axiom 10. Because of extreme wet and dry seasons in a large part of the United States, it is impracticable at present to prevent all moisture-content fluctuations in soil foundations and in soil substructures; but it is practicable to control the fluctuations to such an extent that their ill effects on the superstructure will be reduced, if not eliminated.

In some localities, all of the soil is uniform in type, but is highly expansive, and plastic, clay. Such material must be used as the natural foundation and for building the substructure. In these cases, provisions must be made for maintaining uniform moisture-content fluctuations, as nearly as is practicable, throughout the entire natural foundation and substructure after the pavement is laid. Where the pavement leaks, such as at expansion joints and contraction cracks in concrete pavement, and at cracks and breaks in bituminous topped pavements, the underlying clay soil becomes wet, expands, and therefore raises the pavement more at these concentrated places than elsewhere. As a result, objectionable riding surface irregularities develop.

The development of objectionable riding surface irregularities will be reduced greatly (particularly at points where changes in soil types unavoidably occur) if the upward movements from the original elevations of the pavement which occur during wet seasons are equal to the downward movements from the original elevations of the pavement which occur during droughts. If the upward movements are equal to the downward movements, the variation from the original elevations or smooth riding surface of the pavement will be a minimum. To accomplish this objective, the moisture content, density, and volume of the soil at the time the pavement is laid on it must be the mean of that which subsequently will prevail, as nearly as practicable. If clay soil is compacted to its maximum density with its optimum moisture at the time the pavement is laid, subsequent wetting of the soil during rainy weather will cause the pavement to rise above its original elevations. It will never subside to below its original elevations, and the movement will be a maximum from the original elevations.

If the soil is saturated with water and expanded to its maximum volume when pavement is laid, the pavement will never rise above its original elevations. When the excess moisture is lost during dry weather, the pavement will subside and the amount of movement from the original will be a maximum.

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[4] "Roadbed Design for Self-Drainage," by Henry C. Porter, *Engineering News-Record*, June 16, 1938, Fig. 5.

Suppose one half the length of a highly expansive soil embankment is compacted to its maximum density and the other half is saturated with water (expanded to its maximum volume) at the time the pavement is laid on each half. If extended dry weather follows completion of the work, the first half will not move, the second half will subside, and an irregularity in the riding surface of this pavement will develop at the junction of the two halves. On the other hand, if the completion of the pavement is followed by slow rains which appreciably raise the moisture content in the soil of the first half, this pavement will rise above its original elevations, the pavement on the second half will not move, and an irregularity in the riding surface will develop at the junction of the two halves.

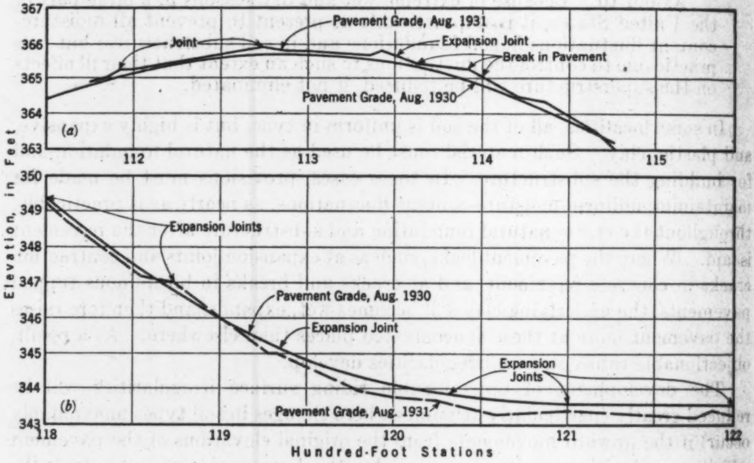

FIG. 8.—CHANGES IN PAVEMENT GRADE

This phenomenon is illustrated in Fig. 8. The lower line of Fig. 8(*a*) shows the profile of the top of a concrete pavement when it was finished in August, 1930. The soil was relatively dry and compacted at the time the pavement was laid, the surface being wet to a depth of 10 to 12 in. There was a great quantity of rain during the following fall and winter, at which time surface irregularities developed. By August, 1931, all this section of pavement had risen above its original elevations—more so at the expansion joints than elsewhere. The upper solid line of Fig. 8(*b*) shows the profile of the top of the pavement on another section of the same project (adjacent to it), when it was finished in August, 1930. The embankment soil on this section was jetted and ponded with water three weeks prior to the laying of the pavement. By August, 1931, this section of pavement had subsided from its original elevations, but not as much at the expansion joints (which perhaps had leaked) as elsewhere.

An expansion joint, without dowels, was the junction of the two projects.[5]

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[5] *Proceedings*, International Conference on Soil Mechanics and Foundation Engineering, Cambridge, Mass., Vol. II, 1936, p. 258; also Figs. 5 and 7.

The natural soil on the two projects was of the same type of highly expansive clay. One pavement was laid during a long period of intermittent slow rains about the first of 1931. At the time this section of pavement was laid, the soil fill was saturated with rain water and perhaps expanded to its maximum volume. Later in 1931, after considerable dry weather, the pavement was laid on the second project when the soil was relatively dry and in a shrunken condition. During the drought of 1934 the pavement on the first project subsided as much as 2.5 in. more than did the pavement on the second project, which caused an irregularity to develop in the riding surface at the junction of the two projects. During the following winter and spring of 1934–1935, there were many slow rains. By June, 1935, the slab-ends lacked only $\frac{5}{8}$ in. maximum of being back to the same elevations.

Where there is non-uniformity of moisture content and density in the clay soils at the time pavement is laid thereon, riding surface irregularities develop both during extreme wet seasons and during droughts.

The foregoing comments have been concerned with compacted, coarse-grained material, such as sand, which does not change in volume with moisture-content fluctuations, and with fine-grained clays which expand in volume when their moisture contents are increased above their shrinkage limits. There is another axiom which also must be taken into consideration—the bulking of sand when it contains a small quantity of moisture and is disturbed or manipulated.

> Axiom 11. When water is added to undisturbed sand until the sand is saturated or inundated, the water tends to compact or decrease the volume of the sand; but when damp sand is manipulated, it bulks.

It has been demonstrated repeatedly that when a quantity of dry sand is stirred while water is added slowly to it, the sand rapidly bulks as more and more water is added, until, with a certain quantity of moisture, the sand reaches a maximum volume. The increase in volume of the sand, when measured in the bulked condition, is much greater than the volume of the water added. The mixing of 5% or 6% of water by weight may cause the sand to increase in volume as much as 20% or even 30%. Further additions of water tend to recompact the sand and decrease its volume as more and more water is added. When sand is saturated, or inundated, the volume of the sand is approximately the same as when it is measured dry. The degree of bulking varies with the type of sand. If the dry sand is not stirred or disturbed while the water is poured on it, the volume of the sand will not change appreciably.

This characteristic of sand should be borne in mind when pavement is placed on a sand fill or blanket. If sand is not perfectly dry or is not entirely saturated with water at the time pavement is placed on it, it is likely to be in a bulked condition. If the sand is bulked, eventually it will very likely become saturated with water under the pavement, settle, and cause the pavement to subside, or leave it suspended on an air pocket or pockets to carry traffic loads by the beam strength of the pavement.

This phenomenon is illustrated by observations made on a project where the plans called for a 12-in. layer of sand blanket to be placed entirely across the crown of clay fills, with concrete pavement to be laid on the sand. For the convenience of hauling materials on the clay shoulders, the sand first was placed in a trench cut in the clay soil to the 20-ft pavement width; the pavement was laid on the sand and was cured by ponding water on it. After the ten-day curing period, when the clay shoulder soil was cut away to the bottom of the sand under the pavement in order to extend the sand blanket to intersections with the roadway side slopes, air pockets were found between the top of the sand and the bottom of the pavement. The sand was wet after being placed in the trench; but afterward it probably was disturbed during fine-grading and shaping operations, and consequently bulked. After the pavement was laid, curing water probably reached the bulked sand and caused it to subside from the bottom of the pavement. If the sand had been wet thoroughly after it was fine-graded and shaped, these settlements in the sand would have developed and been remedied before the pavement was laid.

The foregoing comments have dealt to a large extent with the proper longitudinal design of the soil substructure. The transverse design also is important, particularly in the widening of old pavements. The phenomena are practically the same.

> Axiom 12. The mechanics of the soils must be given proper consideration in the transverse as well as the longitudinal design and construction of the roadway.

Before plans are drawn for pavement widening, the soil structure underlying the old pavement should be investigated thoroughly, its condition determined, and the plans drawn accordingly. For illustration of the importance of the preliminary investigation: In 1937, plans were drawn (without preliminary soil embankment investigations) for widening an old concrete pavement 11 years after the soil embankment was built and 7 years after the 10 ft width was built. When water was ponded on the old shoulder soil in preparing to widen the pavement to the 20-ft width in 1937, non-uniform settlements occurred in the

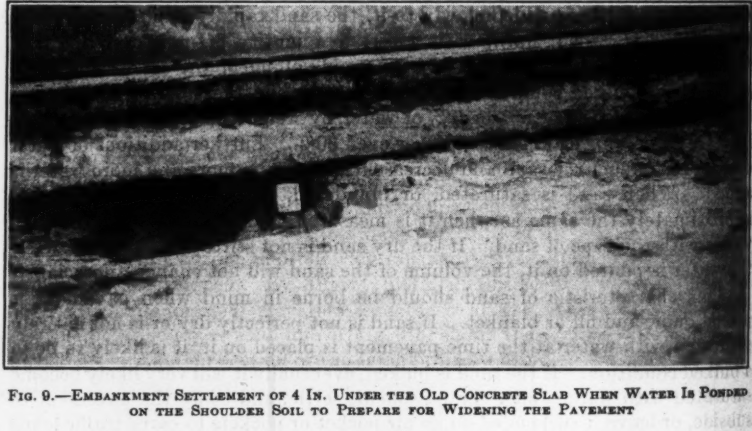

Fig. 9.—Embankment Settlement of 4 In. Under the Old Concrete Slab When Water Is Ponded on the Shoulder Soil to Prepare for Widening the Pavement

soil, as shown in Figs. 9 and 10. These settlements occurred not only in the soil of the shoulders where the new pavement was to be placed, but also under the edge of the old pavement.

If the new width of pavement had been laid on the shoulder soil without ponding water on it first, non-uniform settlements eventually would have occurred and caused cracks and riding surface irregularities to develop in both the new and old pavement. This embankment was sandy soil and was in a bulked condition when the original concrete pavement was laid thereon in 1930.

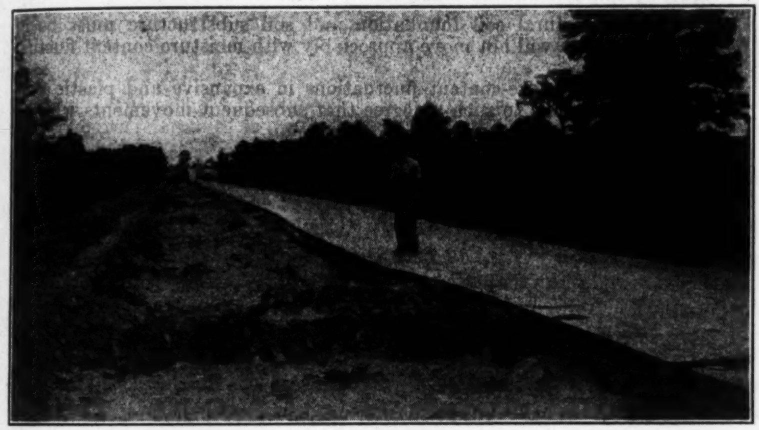

FIG. 10.—ILL EFFECTS OF FOUNDATION SETTLEMENT

If the preliminary investigations show the underlying fill and natural formations beneath the old pavement to be structually sound, the type of soil, moisture content, and density of the soil substructure for the extended width of pavement should be made as nearly like the old part as is practical. An illustration of the subsequent ill effects of a difference in type of construction in the two parts of fills and embankments are illustrated by observations and findings made in 1937. In this instance the old 9-ft width of concrete placed in 1924 is on clay soil. In 1933, when the pavement was widened to the 20-ft width, 12 in. of sand were placed on the old clay shoulder soil and the 11 ft additional width of concrete was placed on the sand. The shoulders of the new concrete were built of clay soil. Subsequent movements in the abutting different types of construction have been different and caused a 2-in. maximum crack to form between the old and new pavement in some places. In this case the major movement differential was transverse instead of vertical. This is one type of riding surface failure caused by lack of uniformity in designing the two parts of the roadway.

During the hot dry weather, the clay shoulder soil on the first of the two projects, previously mentioned[5] subsided as much as 5 in. below the top edge of the pavement. It has been found that when these shoulders are built up with

the same type of clay soil during the dry weather, the shoulder soil re-expands when the rainy season comes until it extends several inches above the top of the pavement and must be cut away. If concrete for widening is laid on this shoulder soil during dry weather, when the soil is in a shrunken condition, the soil eventually will become wet, expand in volume, and raise the new pavement above the old.

> **Axiom 13. In order to build roadways with pavements that will not develop appreciable riding surface irregularities, one or two major items must be embodied in the construction plans:**
>
> **(*a*) The natural soil foundation and soil substructure must be of materials which will not move appreciably with moisture content fluctuations; or,**
>
> **(*b*) The moisture-content fluctuations in expansive and plastic soils must be controlled to such a degree that subsequent movements will not be appreciable.**

DISCUSSION

Victor J. Brown,[6] **Esq.** (by letter).—What Mr. Porter has to say about one's first impression when beginning a study of soil mechanics (see heading "Definitions") is undoubtedly true. Practical usage has done much already to simplify the understanding of basic principles. Continued application of standardized test procedures to practical construction, followed by investigation and report, is also necessary if soil analysts are to reduce the subject to simple terms for more general understanding.

Mr. Porter has proposed a nomenclature which, the writer believes, should be adopted as rapidly as possible. Engineers speak loosely of a "fill," an "embankment," or a "foundation." The writer has many times heard the substructure referred to as the "subgrade." As this term indicates, a subgrade is a grade (a line or a plane) beneath something—a pavement, for example. Depending upon the type of construction, the wearing surface may have a different subgrade from the base, or the subbase.

After reading the thirteen axioms listed by Mr. Porter, the writer is left with one predominant thought; that is, that uniformity is a basic principle in dealing with soils—uniformity in moisture content, or uniformity in soil masses, or uniformity of various types of soils in separate layers. To the extent that uniformity is a desirable condition, investigation, testing, and design are important. In construction, one way to obtain uniformity of soil in the substructure is to mix all soil during excavation in the cut. In a deep cut, this uniformity is easily obtained by shooting the full face of the cut and then loading the loose, mixed material for haul to the embankment substructure.

When the type of construction equipment available can more economically excavate and dump in layers, construction procedure should be such as to place similar soil types in uniform layers, all uniformly compacted. Uniform compaction is as important as placing excavated material in uniform layers, whether compacted with Proctor optimum water content or something less. Desirable density may be obtained with less than Proctor optimum water content provided the proper energy is expended on compaction either with heavier compacting rollers or more passes of the average compacting rollers. In compacting the soil substructure, the writer feels that it can be compacted too densely unless provision is made to positively prevent either increase or decrease of the contained water content—that is, prevent any change in water content after final compaction.

Several contractors have obtained uniform optimum water content by jetting the deposit from which excavated substructure soil is taken. This works particularly well when excavation is taken from borrow pits.

Mr. Porter stated one point, in Axiom 8, which deserves consideration to a much greater extent than merely a mention. The writer was pleased to note that he expressed the desired goal in terms of a desired uniform density rather than expressing compaction in terms of a certain amount of work to be done by a certain type of equipment. Too often specifications stipulate what volume of work is to be done by a particular type of equipment rather than leaving the selection of procedure for obtaining uniform density to the contractor, and specifying a required density. By stipulating what equipment shall be used and how, engineers are throwing obstacles in the path of progress for soil mechanics. Rather, they should specify a required density or state of compaction and allow construction men to devise their own methods for obtaining the required results.

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[6] Publishing Director, *Roads and Streets*, Gillette Publishing Co., Chicago, Ill.

To Axiom 13 the writer would add (*c*) to the effect that uniform compaction to a certain density or to a particular compaction condition must be specified.

A. A. Eremin,[7] Assoc. M. Am. Soc. C. E. (by letter).—In the form of axioms the author has shown various fundamentals of influence of the subgrade soil behavior on a roadway pavement. The axioms would be considerably clarified if their relation to the natural laws of soil mechanics were explained. The object of soil mechanics study, as in any other science, is to develop natural laws and find their limitations.

It is well known that the relation of pressure to the moisture content in soils is somewhat similar to the relation of stress to strain in solid materials. With this conclusion in mind, the author's axioms relating to the influence of the fluctuation in moisture content on deformation of highway slabs may be clarified. Movement and fluctuation of the moisture content in soils follow the hydrodynamic laws. Therefore, the author's axioms relating to displacements and deformations in a saturated soil lying on the impervious sloping strata are obvious.

It is almost impossible to classify soils according to their percentage of sand or clay content. However, familiarity with the characteristic features of sandy and clay soils may help to make interpretations of behavior of soils with various content of sand and clay.

Further development of soil mechanics applied to highway construction will be based on the natural laws of physics and mechanics. Summary of the axioms without clearly determined princeps may be lengthy and difficult for practical application.

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Paper No. 2126

MAXIMUM PROBABLE FLOODS ON PENNSYLVANIA STREAMS

BY CHARLES F. RUFF,[1] M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. JOSEPH L. BENSON, H. ALDEN FOSTER, EDGAR E. FOSTER, C. S. JARVIS, GORDON R. WILLIAMS, EMIL P. SCHULEEN, WALDO E. SMITH, AND CHARLES F. RUFF.

SYNOPSIS

A method, with supporting data, for estimating the maximum probable flood from drainage areas of 100 to 6,000 sq miles is presented in this paper. A maximum storm is derived for various seasons and regions of the state from a study of the records of past storms in Eastern United States. The flood hydrographs caused by 100% runoff from this storm on various sizes of a standard watershed are estimated, and means are developed for correcting these hydrographs to give the corresponding flow from an actual watershed.

A comparison of the maximum flood peaks derived from the storm data with the largest recorded past floods shows a general agreement. Although no frequency is assigned to the flood developed, the basis on which it is derived indicates that such floods must be very rare on any one watershed, and are unlikely to be exceeded.

Although made primarily for Pennsylvania streams, several features of the method are generally applicable. The data used cover a large part of Northeastern United States so that, with modifications required by the locality, they should prove useful in other eastern states.

GENERAL

The term "maximum probable flood" is used to describe not the largest flood possible, but a flood so large that the chance of its being exceeded is no greater than the hazards normal to all of man's activities.

In general, the size of floods at a given point on a stream depends on the watershed—its size and shape; its location with respect to sources of storm rainfall; its imperviousness; and, to some extent, on the slope, pattern, and nature of the stream channels that drain it. These watershed factors remain

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NOTE.—Published in September, 1940, *Proceedings*.

[1] Engr., Federal Power Comm., Washington, D. C.

the same for every flood on the watershed, but differ for different watersheds. The size of any individual flood, however, depends also on the storm rainfall that causes it—its intensity, duration, and the location of the storm center with respect to the watershed. The condition of the ground at the time the storm occurs—whether it is dry, saturated, or snow covered—is also a factor. These storm factors vary with every flood, and their different combinations, which are common to all watersheds, account for the difference in flood size observed on the given stream. The maximum probable flood results from the maximum probable combination of these factors acting on the given watershed. The flood record of a single stream (which in Pennsylvania will be from 20 to 50 years) is too short to insure that such a combination has occurred there. The shortness of a single record, however, may be offset by using records from a large area, since what is rare on a single stream may be a fairly frequent occurrence within a large area. Before such a procedure can be applied, the storm factors, which are common to all watersheds, must be separated from the watershed factors, which are different for each.

The first part of the paper is concerned with estimating how the three types of storms which cause floods in the state vary in size with their location, which determines the location factor, and what would be the maximum probable size of these storms in various parts of the state. Storms are complex phenomena, the quantity of rain varying with both duration and the area covered. They can be simplified, however, by expressing them in terms of the peak flow which they would produce from a "standard watershed" of variable area, but whose other watershed factors (shape, location, etc.) are kept constant. The storm is assumed to fit the watershed perfectly and to produce 100% runoff. This "standard flood" is computed by means of a synthetic unit graph which takes account of the shape adopted for the standard watershed. The resultant flood, when corrected for differences in shape and location between the standard and an actual watershed, and with the application of a suitable runoff coefficient, becomes the maximum probable flood for the actual watershed.

Conversely, actual floods of record can be transformed to equivalent floods from the standard watershed, thus eliminating differences between individual watersheds and permitting comparison of the variable storm factors.

It will be obvious that, in estimating anything as intangible as the maximum probable flood, simplicity and reasonableness are more important than extreme precision. The only certainty in regard to any future storm is that it will not exactly duplicate those of the past nor, for that matter, those developed herein. Thus, certain short cuts and approximate assumptions are used which might be out of place in estimating a past flood, or predicting a flood in process from known rainfall data.

CLASSIFICATION OF FLOOD-PRODUCING STORMS

A brief description of the topography of the state is pertinent. Pennsylvania lies almost entirely in three drainage basins: The Delaware River, the Susquehanna River, and the Upper Ohio River. High mountains (2,000 to 3,000 ft) separate the Ohio and Susquehanna basins. In the north, lower mountains (1,500 to 2,000 ft) separate the Susquehanna and Delaware basins; in the south, the divide between them is not higher than 500 ft. The western and northern quarters of the state are rugged and mountainous in contrast to the relatively level southeastern quarter. The Ohio Basin lies on the western slopes of the Allegheny Plateau and is sheltered from the moisture-bearing winds from the ocean by mountains to the east but is exposed to storms approaching from the level Ohio Valley to the west. The Delaware Basin, on the other hand, and that portion of the Susquehanna Basin below the point where the river cuts through the mountains, lie on their eastern slopes and on the Atlantic Coastal Plain. This latter area is not only nearer the ocean, but is unprotected by mountains from coastal storms and hurricanes. However, it is not accessible to storms from the west until they have first passed over a considerable elevation.

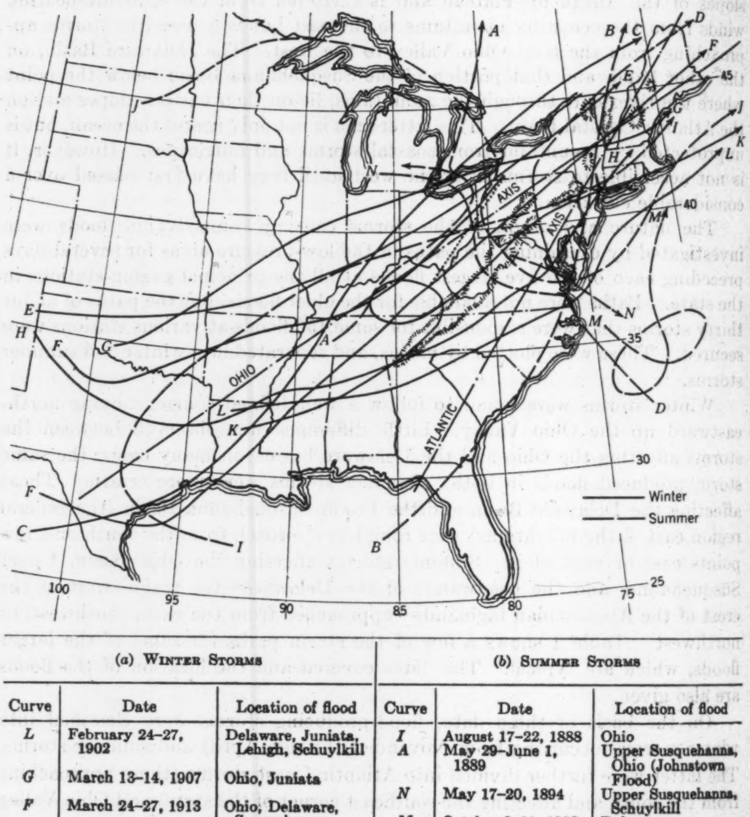
The nature and origin of the storms causing Pennsylvania floods were investigated by examining the paths of the low-pressure areas for several days preceding each of the five largest floods at all the principal gaging stations in the state. Paths were not available for the older floods, but the paths of about thirty storms that were responsible for some 150 floods at various stations were secured. These were plotted by basins, and separated into winter and summer storms.

Winter storms were found to follow a well-defined course, moving northeastward up the Ohio Valley. Little difference was observed between the storms affecting the Ohio and the Delaware basins; in many cases, the same storm produced floods in both. Summer storms were more erratic. Those affecting the Delaware Basin and the Lower Susquehanna Basin (the general region east of the mountains) were found to approach from the south or a few points east or west of it. Summer storms affecting the Ohio Basin, Upper Susquehanna, and the headwaters of the Delaware—the region west of the crest of the Appalachian highlands—approached from the west, southwest, or northwest. Table 1 shows a few of the storm paths for some of the larger floods, which are typical. The dates covered and the location of the floods are also given.

On the basis of these data, flood-producing storms were classified into winter storms (occurring from November through April) and summer storms. The latter were further divided into Atlantic Coast storms (those approaching from the south and affecting the southeast corner of the state) and Ohio Valley storms (those approaching from the west and affecting the northern and western parts of the state). Storms *B*, *D*, and *M*, Table 1, are typical of Atlantic summer storms; and storms *A*, *I*, *J*, and *N* are typical of Ohio Valley summer storms. The winter storms are shown by solid lines.

This classification is consistent with the fact that winter storms depend mostly on frontal action, and the precipitation occurs wherever the warm and cold air masses happen to impinge. Summer storms, on the other hand, although subject to frontal action, are often caused by moist, unstable air being forced upward by passing over hills and mountains, and are thus more subject to the influence of the topography.

TABLE 1.—STORMS THAT CAUSED GREAT FLOODS IN PENNSYLVANIA
(Curves Show Path of Low Pressure Areas; Winter Storms in Solid Lines, Summer Dotted)



(a) WINTER STORMS (b) SUMMER STORMS

Curve	Date	Location of flood	Curve	Date	Location of flood
L	February 24–27, 1902	Delaware, Juniata, Lehigh, Schuylkill	I	August 17–22, 1888	Ohio
G	March 13–14, 1907	Ohio, Juniata	A	May 29–June 1, 1889	Upper Susquehanna, Ohio (Johnstown Flood)
F	March 24–27, 1913	Ohio, Delaware, Susquehanna	N	May 17–20, 1894	Upper Susquehanna, Schuylkill
			M	October 8–11, 1903	Delaware
C	Nov. 15–18, 1927	Delaware, Ohio	B	Sept. 29–30, 1924	Delaware, Schuylkill
E	Dec. 29–30, 1927	Ohio	D	August 22–25, 1933	Lower Susquehanna, Delaware, Schuylkill
H	March 15–19, 1936	Ohio, Susquehanna, Delaware	J	July 4–10, 1935	Ohio, Upper Susquehanna, Upper Delaware, Schuylkill
K	April 24–29, 1937	Ohio			

ESTIMATE OF MAXIMUM PROBABLE DAILY RAINFALL

Atlantic Summer Storms.—Storms similar to the Atlantic summer storms causing Pennsylvania floods occur all along the coastal plain, from Maine to Florida. They are larger in the South and smaller in the North. In order to

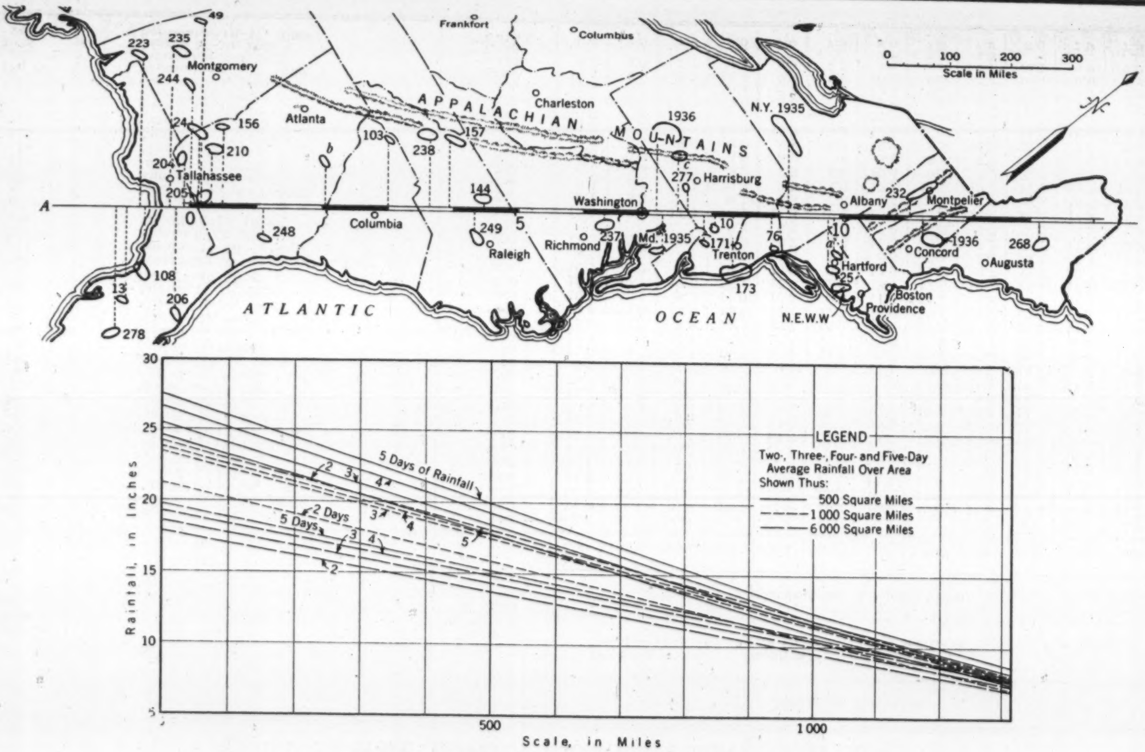


FIG. 1.—STORM RAINFALL ALONG THE ATLANTIC COAST AXIS

TABLE 2.—STORM RAINFALL ALONG THE ATLANTIC COAST AXIS

Designation on Figs. 1 and 2	Date	Distance along axis, in miles	INCHES OF RAINFALL:											
			Duration, Two Days			Duration, Three Days			Duration, Four Days			Duration, Five Days		
			Drainage Areas, in Square Miles:											
			500	1,000	6,000	500	1,000	6,000	500	1,000	6,000	500	1,000	6,000
278	September 4–7, 1933	−110	12.7	12.3	10.8	16.3	15.6	13.6	16.3	15.7	13.7	16.3	15.7	13.7
13	September 24–26, 1894	−90	11.9	11.7	10.5	12.2	11.9	11.2	12.2	11.9	11.2	12.2	11.9	11.2
223	September 18–22, 1926	−85	16.8	15.8	11.9	16.6	15.7	12.0	16.7	15.8	12.1	16.7	15.8	12.2
108	June 29 to July 3, 1909	−60	14.2	13.0	9.9	14.6	13.4	10.5	15.4	14.9	12.0	16.4	16.0	14.2
235	June 2–6, 1928	−30	12.7	12.4	11.1	13.1	12.8	11.8	14.4	14.0	12.6	14.4	14.0	13.1
206	October 7–11, 1924	−25	18.9	16.7	8.9	19.3	17.2	9.9	19.5	17.4	10.7	21.5	19.0	11.6
204	September 13–17, 1924	−10	12.4	11.9	10.1	12.5	12.2	10.5	13.7	13.0	10.5	13.7	13.0	10.9
244	March 12–15, 1929	00	23.5	21.6	15.6	25.5	23.6	17.7	25.5	23.6	18.1	25.5	23.6	18.1
205	September 26–30, 1924	25	10.0	9.2	7.4	10.1	9.4	8.1	10.9	10.4	8.4	11.2	10.7	8.5
24	March 22–23, 1897	25	11.3	11.0	9.6	11.3	11.0	9.6	11.3	11.0	9.6	11.3	11.0	9.6
49	April 15–17, 1900	35	11.7	11.4	10.1	17.2	11.8	11.1	17.2	11.8	11.1	17.2	11.8	11.1
210	January 16–19, 1925	45	8.2	8.1	7.3	10.7	10.4	9.4	11.1	10.8	9.7	11.5	11.1	9.9
156	July 6–10, 1916	55	15.8	14.8	11.3	18.7	17.8	13.6	20.0	19.8	17.4	20.4	19.9	17.7
248	September 23–27, 1929	115	16.9	16.0	12.4	18.3	17.5	12.7	18.6	17.8	13.1	18.8	18.0	13.2
b	July 27–31, 1887	210	12.6	11.9	8.8	14.4	13.7	10.9	14.5	13.8	11.2	15.1	14.5	12.2
103	August 24–26, 1908	300	12.0	11.0	8.0	13.9	13.1	11.3	13.9	13.1	11.3	13.9	13.1	11.3
238	August 14–18, 1928	355	10.7	10.4	8.2	11.0	10.5	8.3	11.8	11.1	8.8	11.8	11.1	8.8
157	July 14–16, 1916	390	18.4	17.1	12.5	18.4	17.3	12.7	18.4	17.3	12.7	18.4	17.3	12.7
249	September 29 to October 3, 1929	435	10.7	10.2	9.3	10.8	10.3	9.1	12.9	12.2	9.7	12.9	12.3	9.7
144	October 14–15, 1914	450	8.5	7.5	5.5	8.5	7.5	5.5	8.5	7.5	5.5	8.5	7.5	5.5
237	August 9–13, 1928	630	11.2	10.6	8.4	11.2	10.7	8.8	11.3	10.7	8.8	12.1	11.6	9.3
1936	March 16–19, 1936	710	....	....	....	6.3	6.0	5.0	7.0	6.9	6.0	7.0	6.9	6.0
Md 35	September 5–6, 1935	730	13.9	13.4	11.4	15.2	14.6	12.4	15.4	14.7	12.4	15.4	14.7	12.4
c	May 31 to June 1, 1889	740	9.1	8.8	7.5	9.1	8.8	7.5	9.1	8.8	7.5	9.1	8.8	7.5
277	August 20–24, 1933	750	9.3	8.7	7.1	10.8	9.9	8.0	12.1	11.7	10.0	12.2	11.8	10.0
171	July 19–23, 1919	785	6.1	5.5	4.3	8.2	7.3	4.9	10.5	9.5	6.2	10.7	9.7	7.0
10	May 18–22, 1894	810	7.8	7.2	6.8	8.2	8.0	6.8	8.6	8.4	7.2	9.5	9.1	8.1
173	August 13–17, 1919	830	9.0	8.7	6.6	9.0	8.8	6.7	9.1	8.8	6.9	9.9	9.6	7.7
76	October 8–9, 1903	880	11.9	10.9	8.4	11.9	10.9	8.4	11.9	10.9	8.4	11.9	10.9	8.4
NY 1935	July 7–11, 1935	915	10.0	9.8	7.5	....	....	....	12.9	11.9	8.4	....	....	....
25	July 12–14, 1897	975	9.5	9.1	6.9	9.6	9.3	7.0	9.6	9.3	7.0	9.6	9.3	7.0
a	October 3–4, 1869	980	10.4	9.7	7.8	10.4	9.7	7.8	10.4	9.7	7.8	10.4	9.7	7.8
NEWW	September 16–17, 1932	1,005	10.4	9.0	6.7	10.4	9.0	6.7	10.4	9.0	6.7	10.4	9.0	6.7
232	November 3–4, 1927	1,110	9.1	8.9	7.7	9.1	8.9	7.7	9.1	8.9	7.7	9.1	8.9	7.7
1936	March 16–19, 1936	1,135	....	....	....	....	....	....	7.7	6.7	4.5	....	....	....
268	September 16–17, 1932	1,315	8.0	7.9	7.2	8.0	7.9	7.2	8.0	7.9	7.2	8.0	7.9	7.2

display this variation in size, storm profiles were plotted. This method was explained by the writer in 1938. It consists of plotting the rainfall for various conditions of duration and area against the distance along a base line to the point where the storm occurred. The base line used for Atlantic summer storms, designated as the Atlantic Coast Axis, was drawn parallel to the coast and through the center of the area covered by these storms. All the storms available in the "Miami Report" plus a number of more recent storms occurring in and near Pennsylvania were located on a map (see Fig. 1, and Item (2) in the Appendix), and projected on to the axis. The location is shown by the position of the highest isohyetal line.

Table 2 shows the position and the average precipitation of these storms for the durations and areas used. These storms were all plotted as shown for the 4-day storm on 6,000 sq miles, in Fig. 2. The 1-day rainfall was omitted because of the uncertainty as to the actual duration of the rain. The largest storms plotted on these figures showed a marked and fairly uniform decrease in size as they occurred farther to the northeast. Enveloping lines were drawn to pass over the largest storms on the profiles for each condition of area and duration. Straight lines were found to be satisfactory.

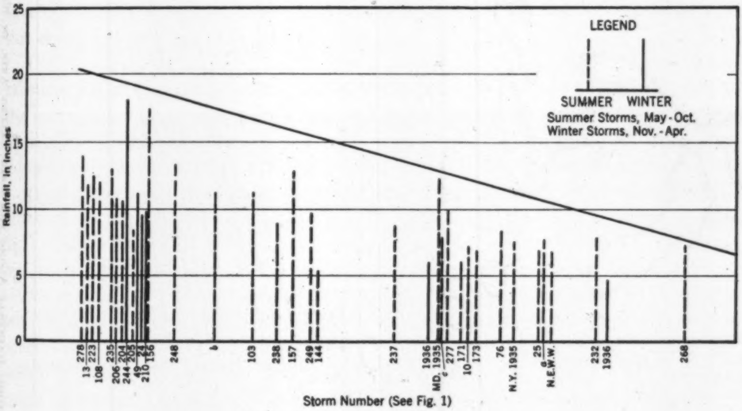


FIG. 2.—ATLANTIC COAST AXIS; 4-DAY STORM RAINFALL ON 6,000-SQ-MILE AREA

The lines for different conditions were adjusted slightly to be consistent with each other—that is, to make the 3-day rainfall more than the 2-day rainfall, and the 500-sq-mile average more than the 1,000-sq-mile average, etc. For this reason, some of the lines pass above all the storms, as in Fig. 2. The enveloping lines for all the conditions of area and duration considered are combined in Fig. 1. Although made for Atlantic summer storms, all available winter storms in the same area were included. Only one winter storm in the far South, No. 244, March, 1929 (storm numbers correspond to those used in the "Miami Report"; see Item (2), Appendix), has any effect on the position of the

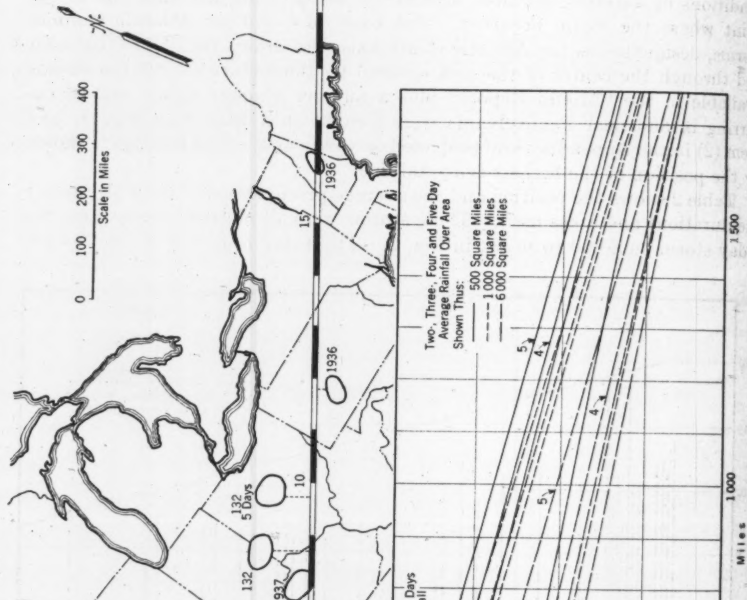


FIG. 3.—STORM RAINFALL ALONG THE OHIO VALLEY AXIS, IN WINTER

enveloping lines. Storm No. 232, the 1927 storm in Vermont, was counted as a summer storm, being distinctly of that type, although it occurred on November 3 and 4.

If the enveloping lines were determined from two storms only, their slope might be ascribed to a chance difference in size between the two storms. However, when several storms at different points support the lines, and others approach them, it suggests a limit to the maximum size of the storms at various points. The trend of decreasing size as the storms occur farther north is thought to be due to the greater distance traveled by the moist air from its source in the tropical Atlantic Ocean or Caribbean Sea. The farther it travels, the greater will be the chance of encountering conditions that will precipitate some or all of its moisture.

The size of these Atlantic storms varies with the season of the year. If the size of a storm is expressed as a percentage of the rainfall indicated by the enveloping line, the effect of its location up or down the coast is eliminated and this seasonal variation is emphasized.

Ohio Valley Storms.—For winter storms causing floods in Pennsylvania, a base line called the Ohio Valley Axis was chosen to represent the mean path of the winter storms, as shown in Table 1. All of the winter storms available (see Appendix) which occurred in the region traversed by these paths were considered, and their location plotted in Fig. 3. It is reasonable to suppose that any one of these storms under different conditions might have precipitated its moisture as it passed over the state. The more erratic Ohio summer storms did not show any well-defined average path, and for convenience the same axis was used for them also. The location of the summer storms considered is shown in Fig. 4. It should be noted that the exact position of the axis selected does not materially affect the result, at least so far as Pennsylvania is concerned. The storm data for both winter and Ohio summer storms are shown in Table 3, and were plotted in the manner illustrated for the 4-day storm on 6,000 sq miles, in Fig. 5. The procedure was the same as in Table 2 and Fig. 2 for Atlantic summer storms. In Table 3 and Fig. 5, however, the summer and winter storms are treated separately.

These storms show the same general trend of decreasing size as the storm occurs farther to the northeast. For the summer storms, however, there is a group near the middle of the axis significantly lower than those at the ends. This is probably due to the distance of this part of the axis from a source of moisture—the Gulf of Mexico or the Atlantic Ocean—and the sheltering effect of the mountains between it and the coast. The summer storms tend to decrease in size as they occur farther from the Gulf of Mexico until a point is reached near enough to the ocean to secure moisture from that source. They then increase as the ocean is approached and finally decrease again as the storms occur farther north along the coast, as shown on the Atlantic Coast profile. This has been interpreted by three series of straight lines enveloping all the storms, although if more data were available it is probable that the breaks in the profile would not be abrupt, but rounded off to some extent. Some of the storms shown for the Atlantic Coast Axis are duplicated in Fig. 5, since its

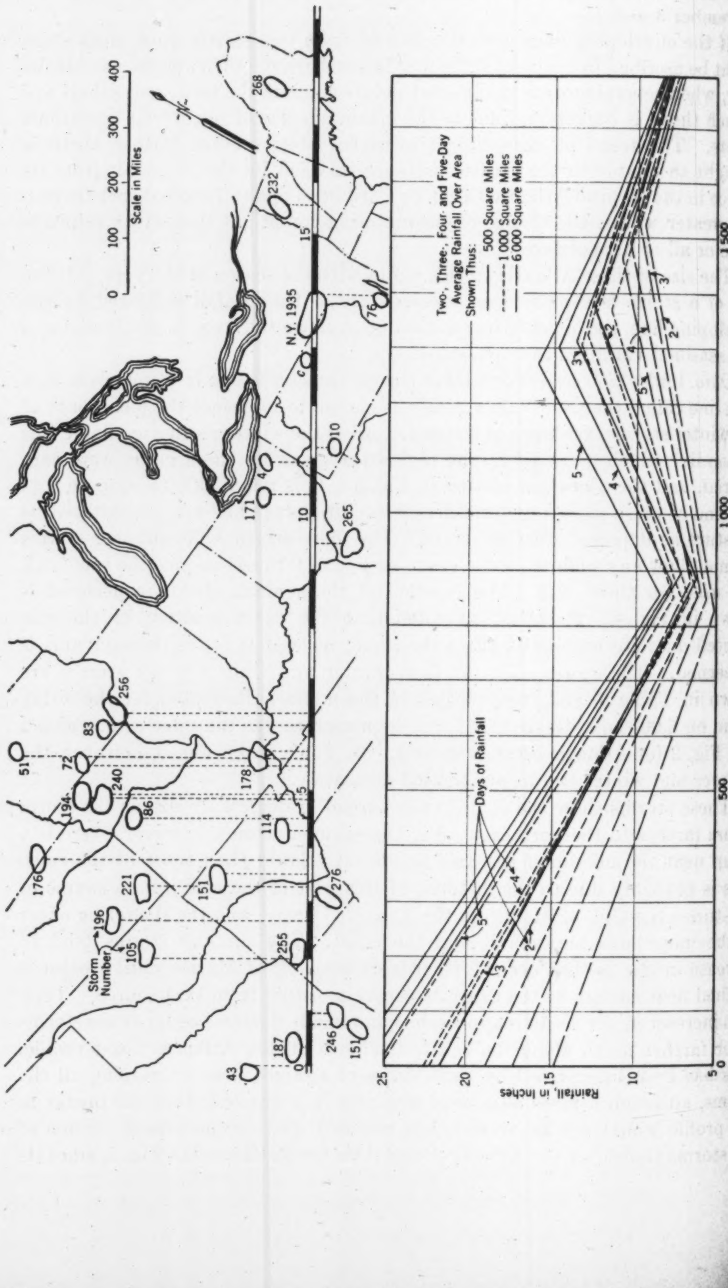


FIG. 4.—STORM RAINFALL ALONG THE OHIO VALLEY AXIS, IN SUMMER

TABLE 3.—STORM RAINFALL ALONG THE OHIO VALLEY AXIS

(*a*) WINTER STORMS

Designation on Figs. 3, 4, and 5	Date	Distance along axis, in miles	INCHES OF RAINFALL:											
			Duration, Two Days			Duration, Three Days			Duration, Four Days			Duration, Five Days		
			Drainage Areas, in Square Miles:											
			500	1,000	6,000	500	1,000	6,000	500	1,000	6,000	500	1,000	6,000
213[a]	Nov. 4–8, 1925	−50	5.4	5.0	...	9.5	10.2	7.9	10.7	10.3	8.9	10.9	10.3	9.1
136	Dec. 2–5, 1913	0	11.2	10.5	8.4	13.3	12.8	10.9	14.4	13.8	11.5	14.4	13.8	11.5
15	Dec. 17–20, 1895	235	9.3	9.0	7.7	10.7	10.5	9.2	11.0	10.5	9.3	11.0	10.5	9.3
228	April 12–16, 1927	380	8.9	8.5	7.4	16.3	15.0	10.4	16.4	15.0	10.6	16.5	15.3	12.2
253	Jan. 7–10, 1930	400	7.8	7.5	6.8	9.8	9.6	8.5	10.1	9.9	9.1	10.1	9.9	9.1
242	Nov. 14–17, 1928	420	10.9	10.6	9.3	10.9	10.6	9.4	11.1	10.8	9.5	11.1	10.8	9.5
63	Mar. 26–29, 1902	470	9.0	8.9	8.4	10.3	9.7	8.7	10.6	10.1	9.2	10.6	10.1	9.2
93	Nov. 17–21, 1906	530	9.9	9.5	7.9	11.7	11.1	9.8	14.8	14.0	11.4	17.8	17.0	14.1
1937[b]	Jan. 20–24, 1937	555	...	...	...	11.2	11.0	9.8	...	...	...	14.4	14.1	13.0
168	Mar. 14–17, 1919	580	10.0	9.5	8.3	10.6	10.3	8.6	11.5	11.0	9.0	11.5	11.0	9.0
130	Jan. 10–12, 1913	600	6.8	6.6	5.8	7.5	7.4	6.9	7.5	7.4	6.9	7.5	7.4	6.9
208	Dec. 5–9, 1924	660	8.0	7.9	6.8	8.0	7.9	6.9	8.0	7.9	6.9	9.1	8.8	7.6
245	Mar. 21–23, 1929	720	9.5	8.9	7.0	9.5	8.9	7.0	9.5	8.9	7.0	9.5	8.9	7.0
1937[b]	Jan. 20–24, 1937	800	9.1	8.9	8.1	9.8	9.7	8.9	...	...	...	13.0	12.9	11.7
132	Mar. 23–27, 1913	870[d]	8.8	8.4	7.8	9.7	9.5	8.7	10.4	10.0	9.3	10.9	10.8	9.9
1936	Mar. 16–19, 1936	1,180	...	...	...	6.3	6.0	5.0	7.0	6.9	6.0	7.0	6.9	6.0
1936	Mar. 16–19, 1936	1,610	...	...	...	...	...	...	7.7	6.7	4.5	...	...	...
	(*b*) SUMMER STORMS													
175[a]	Sept. 14–16, 1919	−225	11.1	10.8	8.4	11.4	11.1	9.0	11.4	11.1	9.0	11.4	11.1	9.0
135[a]	Oct. 1–2, 1913	−80	13.7	13.3	10.2	13.7	13.3	10.2	13.7	13.3	10.2	13.7	13.3	10.2
43	June 27 to July 1, 1899	0	16.6	14.6	12.3	25.6	22.1	17.5	26.1	22.4	18.4	26.1	22.4	18.4
187	Sept. 7–10, 1921	40	21.1	20.0	14.2	21.1	19.8	14.2	21.8	20.5	14.4	21.8	20.5	14.4
151[b]	Aug. 17–20, 1915	70	16.2	15.0	11.7	19.6	19.5	17.9	19.6	19.5	17.9	19.6	19.5	17.9
246	May 27–31, 1929	100	10.8	10.6	8.5	11.5	11.1	8.9	11.5	11.2	9.1	11.6	11.2	9.5
105	Oct. 20–24, 1908	195	11.0	10.7	8.8	12.6	12.2	10.4	14.2	13.6	11.7	14.4	13.9	11.7
255	May 15–19, 1930	210	7.8	7.5	5.8	11.0	10.6	8.6	12.0	11.8	10.3	12.0	11.8	10.4
196	June 7–11, 1923	270	7.5	6.9	5.0	7.5	6.9	5.5	7.6	7.4	6.2	7.7	7.4	6.3
276	July 23–27, 1933	310	7.1	6.9	6.0	11.1	10.9	9.8	16.6	16.1	14.3	19.4	18.9	16.3
221	Sept. 12–15, 1926	325	8.3	7.5	5.0	8.3	7.5	5.3	8.3	7.7	5.8	8.3	7.7	5.8
151[b]	Aug. 17–20, 1915	350	10.2	9.7	8.3	13.1	12.6	10.7	13.9	13.2	11.2	13.9	13.2	11.2
176	Sept. 17–21, 1919	375	8.8	8.6	7.5	8.8	8.6	7.5	8.8	8.6	7.5	8.8	8.7	7.6
114	Oct. 4–6, 1910	435	11.0	10.2	8.2	13.7	12.8	11.4	13.7	12.8	11.4	13.7	12.8	11.4
86	Sept. 15–19, 1905	460	8.1	8.0	7.1	10.3	10.1	8.6	11.3	10.9	9.4	12.5	12.0	10.0
194	July 8–12, 1922	480	10.5	9.3	5.9	11.4	10.2	6.8	11.5	10.2	7.0	11.6	10.2	7.0
240	Sept. 10–14, 1928	490	6.8	6.5	5.7	7.4	7.2	6.3	7.8	7.5	6.8	8.9	8.5	7.7
72	Aug. 25–28, 1903	545	11.8	10.8	7.7	12.2	11.4	8.7	12.4	11.7	8.8	12.4	11.7	8.8
51	July 14–16, 1900	565	11.7	10.7	8.0	12.2	11.4	8.9	12.2	11.4	8.9	12.2	11.4	8.9
178	Oct. 26–30, 1919	575	9.5	9.3	8.2	10.4	10.1	8.9	10.7	10.5	9.3	11.1	10.7	9.5
88	June 9–10, 1905	625	10.9	10.0	6.9	(One Day Storm)								
256	June 11–15, 1930	640	8.5	8.0	5.8	9.3	8.6	6.2	9.3	8.6	6.3	9.3	8.6	6.3
265	July 4–6, 1932	950	...	...	...	7.5	7.0	3.0	...	...	...	...	...	...
171	July 19–23, 1919	1,035	6.1	5.5	4.3	8.2	7.3	4.9	10.5	9.5	6.2	10.7	9.7	7.0
4	May 15–17, 1893	1,090	...	...	...	8.0	7.9	6.5	...	...	...	...	...	...
10	May 18–22, 1894	1,120	7.8	7.2	6.8	8.2	8.0	6.8	8.6	8.4	7.2	9.5	9.1	8.1
c	May 31 to June 1, 1889	1,275	9.1	8.8	7.5	9.1	8.8	7.5	9.1	8.8	7.5	9.1	8.8	7.5
NY 1935	July 7–11, 1935	1,350	10.0	9.8	7.5	...	...	...	12.9	11.9	8.4	...	...	...
76	Oct. 8–9, 1903	1,395	11.9	10.9	8.4	11.9	10.9	8.4	11.9	10.9	8.4	11.9	10.9	8.4
232	Nov. 3–4, 1927	1,560	9.1	8.9	7.7	9.1	8.9	7.7	9.1	8.9	7.7	9.1	8.9	7.7
268[c]	Sept. 16–17, 1932	1,770	8.0	7.9	7.2	8.0	7.9	7.2	8.0	7.9	7.2	8.0	7.9	7.2

[a] Located in Texas—not shown in Figs. 3 and 4. [b] These storms had two centers. [c] Located in Maine—not shown in Fig. 5. [d] 3-day center; 5-day center moved to 975 miles.

purpose is to show the largest storms in the region covered and not simply the Ohio Valley storms.

The winter storms decrease in size more uniformly and do not show a dip near the center of the axis, where they are larger than the summer storms. Apparently, they vary in size with their distance from the Gulf; the horizontal enveloping lines to the left of Figs. 3 and 5 cover a group of storms roughly equidistant from the Gulf. This part of the profile has little significance for Pennsylvania, and might be revised in a study made for that region. The enveloping lines, as drawn, are consistent with the relative freedom of winter storms from the effect of topography and the absence of large winter storms along the Atlantic Axis. The Ohio Axis profiles are not as clear-cut as the Atlantic, especially the summer profiles, and certain sets of the plotted data, taken by themselves, appear arbitrary. However, when the entire group of lines is considered together, it is apparent that the lines, while consistent as a group, are in many cases determined by different storms for different conditions of area and duration. No storm, in fact, equals the envelop for every such condition. The combined enveloping lines are shown in Figs. 3 and 4. A plotting of seasonal occurrence of the Ohio Axis storms showed no marked seasonal variation in size aside from the difference between summer and winter storms indicated by the storm profiles.

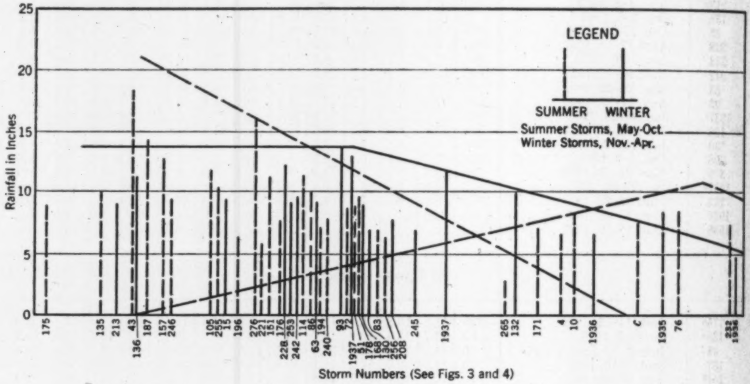
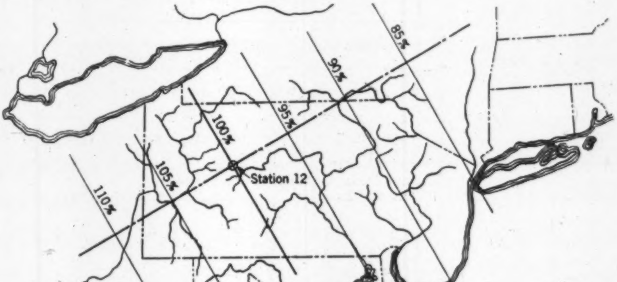


FIG. 5.—OHIO VALLEY AXIS; 5-DAY STORM RAINFALL ON 6,000-SQ-MILE AREA

Variation of Size with Location.—The effect of location on different conditions of area and duration seems to be about the same. If, for example, the 4-day, 6,000-sq-mile rainfall at one point is half that at another point, the 2-day, 500-sq-mile rainfall will also be approximately half. If this were exactly so, all the sets of lines on Figs. 1, 3, and 4 would intersect at common points on the base lines. A study of maximum rainfall at single stations over the area covered by the storm profiles led to the conclusion that the small area, or local storms, which might not have been included in the "Miami Report," also varied with location in about the same manner as the large area storms. It was assumed, therefore, that at least within the state location has the same effect on all conditions of area and duration. The size of the maximum probable rainfall at any point may thus be expressed as a percentage of the maximum probable rainfall at some fixed point of reference, and this percentage may be taken as a constant for all conditions of duration and area. For this purpose, station 12, Ohio Axis, was chosen as the point of reference for storms referred to that axis, and station 8, Atlantic Axis, for Atlantic summer storms.

The variation in size of the maximum probable storm, expressed in percentages of the size of the storm at these reference points, is shown in Fig. 6.



Note: Percentages Refer to Storm at Station 12 Ohio, or Station 8 Atlantic Axis, as the Case May be

FIG. 6.—VARIATION IN SIZE OF STORM WITH ITS LOCATION

In drawing the winter map, the percentage lines are run at right angles to the axis, thus giving the same values to points having approximately the same distance of travel from the Gulf. For the summer storms the boundary between the Ohio Valley and Atlantic Coast storms was assumed to follow the crests of the mountains, but the latter are not, perhaps, a definite limit to either type of storm. In order to reconcile the values determined along this boundary from the two different axes, it was necessary to bend the lines away from a right-angle position, giving due regard to the large storms on either side of the axis. The resulting map, of which only the Pennsylvania part is shown, was roughly similar to maps for single-station rainfall shown by the late David L. Yarnell, M. Am. Soc. C. E. (Item (4), Appendix) and the "Miami Report" (Item (3), Appendix).

TABLE 4.—NUMBER OF STORMS TOUCHING THE PROFILE AND THOSE EXCEEDING 90% OF IT DURING 50 YEARS OF RECORD

Duration of storm, in days	WATERSHED AREAS, IN SQUARE MILES:							
	500		1,000		6,000		Total	
	Percentage of Envelop Storm:							
	100	90 to 100	100	90 to 100	100	90 to 100	100	90 to 100
(*a*) SUMMER STORMS, ATLANTIC COAST AXIS								
2	1	7	3	7	1	2	5	16
3	3	6	3	3	1	3	7	12
4	0	4	3	3	0	4	3	11
5	0	4	2	3	0	2	2	9
Total conditions	4	21	11	16	2	11	17	48
Total number of storms involved	....	....	....	....	....	....	3	9
Average number of conditions per storm	....	....	....	....	....	....	5.67	5.33
Average interval between storms, in years	....	....	....	....	....	....	17	5
(*b*) WINTER STORMS, OHIO VALLEY AXIS								
2	1	6	2	5	2	6	5	17
3	0	1	0	1	2	4	2	6
4	0	2	1	2	2	6	3	10
5	1	2	1	2	2	4	4	8
Total conditions	2	11	4	10	8	20	14	41
Total number of storms involved	....	....	....	....	....	....	7	13
Average number of conditions per storm	....	....	....	....	....	....	2	3.15
Average interval between storms, in years	....	....	....	....	....	....	7	4
(*c*) SUMMER STORMS, OHIO VALLEY AXIS								
2	4	6	3	6	5	7	12	19
3	2	3	2	8	3	8	7	19
4	2	4	2	7	1	5	5	16
5	0	3	1	2	1	3	2	8
Total conditions	8	16	9	23	10	23	26	62
Total number of storms involved	....	....	....	....	....	....	11	15
Average number of conditions per storm	....	....	....	....	....	....	2.36	4.13
Average interval between storms, in years	....	....	....	....	....	....	5	3

Discussion of Storm Frequency.—The records used in drawing the storm profiles cover, roughly, the period from 1892 to 1937 (see Appendix). Three of the largest storms before 1892 are included, but a number of storms in the early part of the record have probably been missed. For the purpose of discussing the frequency of these storms, it will be sufficiently accurate to assume that the record covers 50 years. A storm, therefore, whose rainfall equals the amount shown by the enveloping lines at the point where it occurs may be considered to be as large, relative to its position, as any storm of which there is any record during the past 50 years.

Table 4 shows the number of times the enveloping lines have been equaled, and the number of times 90% of the lines have been exceeded for various conditions of duration and area. Of the Atlantic Coast summer storms, three equal the line for seventeen conditions of duration and area, an average of 5.67 conditions for each storm. Since these storms are based on a 50-yr record, the average interval between them may be taken as 17 years. A storm equaling the lines for five or six conditions may thus be expected every 17 years, on the average. Table 4 shows the frequencies with which such storms happen somewhere within the area covered by the records used in drawing the storm profiles. No two of the 100% storms have occurred at the same point, or even opposite the same station on the axis. The irregular distribution of the smaller storms is less significant than the more uniform spacing of the large ones which define the enveloping lines. If it is assumed that a storm equaling the lines on the storm profiles (similar to Figs. 2 and 5) is equally likely to occur at any point along the axis, the chance of a storm occurring in the state can be estimated from the relative size of the areas involved. The total area covered by the Atlantic Summer storms is about 400,000 sq miles. Of this area, 13,000 sq miles lie in the Lower Susquehanna and Delaware basins of Pennsylvania subject to such storms. Thus, there is about one chance in thirty that a storm occurring somewhere in the entire area will occur in the state. If one storm in thirty occurs in the state, the average interval between such storms will be 30 times 17 years or 510 years. The Ohio storms, both summer and winter, covered an area of roughly 780,000 sq miles. On the same basis the approximate frequency of the storms within Pennsylvania is as shown in Table 5.

TABLE 5.—COMPUTED AVERAGE INTERVAL BETWEEN OCCURRENCES OF STORMS IN PENNSYLVANIA

Drainage basin	Storm type	Area, in square miles	Season	INTERVAL, IN YEARS 100% storms	INTERVAL, IN YEARS 90% storms
Lower Susquehanna and Delaware........	Atlantic	13,000	Summer	510	165
Upper Susquehanna and Ohio............	Ohio	32,000	Summer	120	80
Entire state..........................	Ohio	45,000	Winter	120	70

These data are not presented as a precise estimate of the frequency of the storms in question. However, they do indicate the rarity of such storms within the state. The chance of such a storm scoring a "bull's-eye" on a particular watershed is obviously even smaller, and the interval between such occurrences on the same watershed would be very much longer than is indicated in Table 5. The foregoing reasoning suggests how data from a wide area may be substituted for the long records (which are not available) to estimate the 100-yr or 1,000-yr flood, provided that the law of variation within the area is understood.

It is realized that the records are incomplete in several respects, and that 50 years is not a long time. It is always possible for past records to be exceeded; but even so, the chance that a storm which equals the storm profile will occur on a particular watershed in the position which causes the greatest flood is very small; and the chance that this storm should also happen to be one of the rarer storms that exceed the profiles for just the particular area and duration which are critical for that watershed must be so small as to be considered improbable. The quantities of rainfall shown in Figs. 1, 3, and 4, therefore, were used as the daily rainfall of the maximum probable storm for the area and season.

MAXIMUM PROBABLE RAINFALL FOR SHORT DURATIONS

An estimate of correspondingly large rainfalls for durations shorter than 2 days was then made. The relation between rainfall of the same relative size (or frequency) but of different durations is shown in Fig. 7(*a*), in which the 100-yr rainfall, in inches, at station 12, Ohio Axis, as taken from the maps in the reports of Mr. Yarnell and the Miami Conservancy District, is plotted against duration. The data were not separated into summer and winter, but the summer storms fix the position of most points. Maximum probable summer rainfall at station 12 for 2 to 5 days, as given by the profiles on Figs. 3 and 4, was then plotted in Fig. 7(*a*). On the assumption that the maximum short-duration rainfall would be in the same ratio to the 100-yr rain as are the longer durations, a line parallel to the 100-yr line was drawn through these points. A number of record rainfalls from the "Morgan Report" (see Item (5), Appendix) were also plotted in Fig. 7(*a*) as a check. The line is seen to pass over, or near, all the points except a 1-hr storm in Cincinnati, Ohio, in 1921.

For the corresponding curve for winter, much less data are available. A search of the hourly records of recent years disclosed nothing greater than 1.5 in. in an hour, but the record is too limited to assume that this approaches the maximum. Therefore, the winter line was drawn simply as an extension of the points from the profiles for durations of 2, 3, 4, and 5 days and with a steeper slope than the summer line, indicating a more nearly uniform rain, which is a characteristic of winter storms.

In Fig. 7(*a*), only the rainfall at a single station was considered. In Fig. 7(*b*), the effect of the size of the area on the average rainfall is introduced. These curves show the maximum rainfall for the period, averaged over the area indicated. Using the rainfall at one station for various periods from 1 hr up, from Fig. 7(*a*), and the 2-day to 5-day rainfall over various areas as taken from the storm profiles, the other values are extrapolated. It is realized that this procedure is approximate; it is resorted to only through lack of better data. For Ohio winter rainfall, the same process was followed in preparing Fig. 7(*c*).

Fig. 8(*a*), for summer rainfall at station 8, Atlantic Axis, corresponds to Fig. 7(*a*) for the Ohio Axis, and was constructed in the same manner. In this case, a number of rainfalls in the Morgan report exceed the assumed maximum line. All these prove to be very old records, the most recent being at

Chester, Pa., in 1843. Although these old records are not necessarily in error, the absence of any comparable records in the past 100 years indicates that such storms, if they do occur, are too rare to be termed "probable" for a particular watershed. A few unofficial records, made in buckets, etc., of the 1935 New York storm also exceed the line, but only by a small quantity. The data are extrapolated to various areas and durations in Fig. 8(*b*) in the same manner as for the Ohio Axis.

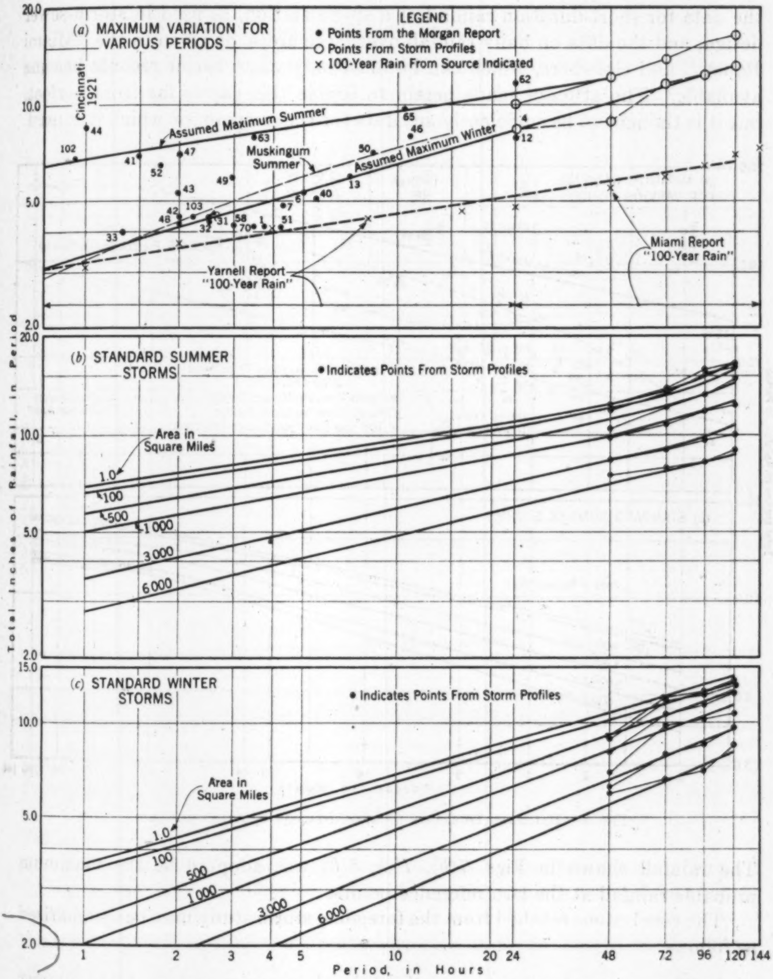


FIG. 7.—RAINFALL DURATION CURVES, STATION 12, OHIO AXIS

The values determined by this extrapolative process which have the greatest error are those for short durations on large areas. However, as the area of the watershed to which the storm is to be applied increases, the importance of the intensity of the rainfall of shorter duration diminishes. There is a gap between the data for short-duration rainfall at a single station, as used in storm-sewer design, and the data on daily rainfall over large areas, as given in the "Miami Report" and elsewhere, which can be filled only when better records become available. The attempt made herein to bridge this gap is far from perfect, but it is thought to be sufficiently accurate for the purpose for which it is used.

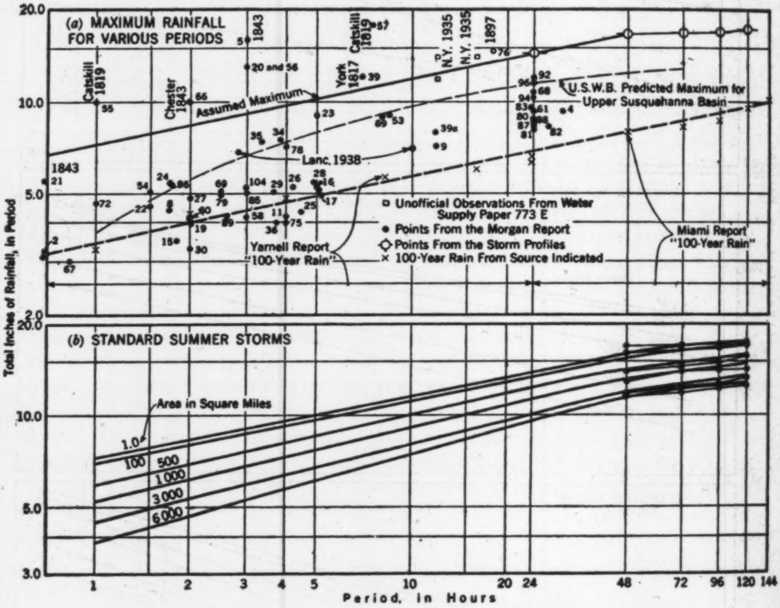


FIG. 8.—RAINFALL DURATION CURVES, STATION 8, ATLANTIC AXIS

The rainfall shown in Figs. 7(*b*), 7(*c*), 8(*b*) was adopted as the maximum probable rainfall at the two reference points.

The conclusions reached from the foregoing storm study may be summarized as follows:

1. Three general storm types—the Atlantic summer and the Ohio summer and winter—are responsible for Pennsylvania floods, and should be treated separately.

2. The occurrence, on a given watershed, of a storm equaling the quantities indicated by the storm profiles (see Figs. 1, 3, and 4) is sufficiently rare to justify their use as a measure of the size of the maximum probable storm.

3. Each type of maximum probable storm varies in size from place to place, as indicated on the profiles, but each may be considered to have the same structure—that is, the same average relation between the different areas and durations, at least throughout the area of the state to which it is applicable.

Development of the Standard Floods

In reducing the aforementioned rainfall estimate to flood flows on various areas, use is made of three devices—(*a*) the standard storm, (*b*) the standard watershed, and (*c*) the standard flood—which are defined as follows:

(*a*) The standard storm is one that occurs at a particular reference point and equals the maximum probable rainfall at that point for every condition of area and duration. The rain is assumed to occur symmetrically throughout the duration of the storm with the maximum hour in the middle, the maximum 2 days a day before and a day after the middle, etc., as shown in Fig. 9, where the rainfall on the fourth and fifth days is symmetrical with the second and first days, respectively. Three standard storms are used: (*a*) Winter and (*b*) Ohio Valley summer, both at station 12, Ohio Axis, and (*c*) Atlantic summer, at station 8, Atlantic Axis.

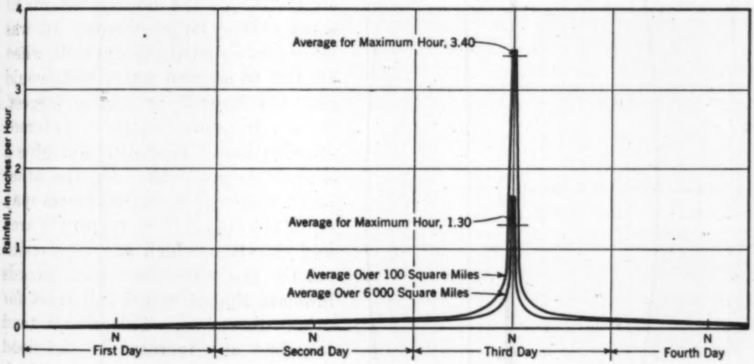


Fig. 9.—Standard Storm (Winter); Station 12, Ohio Axis

(*b*) The standard watershed is one that has constant shape and physical characteristics, but variable size. An ellipse, with a ratio of length to average width of four, was used as being an approximation of the average watershed in the state. The length of the stream corresponds to the length of the ellipse (see Fig. 10).

(*c*) The standard flood is produced by 100% runoff from a standard storm occurring on a standard watershed in such a position that the area of greatest rainfall coincides with the watershed. Three types of standard floods are considered, corresponding to the three types of storms. Each type has a series of values for the peak flow corresponding to various areas of the standard watershed. Since all standard watersheds are identical, except in size, the standard flood is a convenient means of expressing the size, and other characteristics, of the standard storm in terms of flow from various areas.

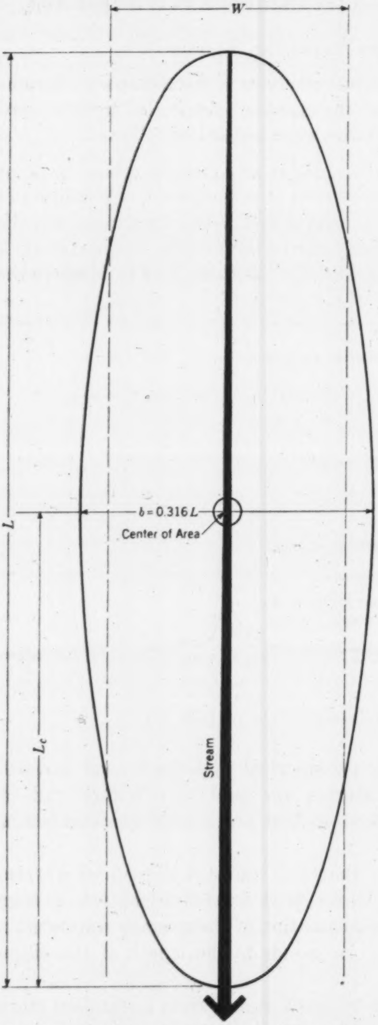


Fig. 10.—Dimensions of Standard Watershed

The Standard Storm.—The standard storms are developed directly from Figs. 7(*b*), 7(*c*) and 8(*b*). The winter standard storm for both the 100-sq-mile and the 6,000-sq-mile average rainfall is shown in Fig. 9. The other types of standard storms, not illustrated, are very similar.

This distribution in time of the standard storms was not assumed to represent typical or average conditions in actual storms, although it is suggestive of the hourly records of some of the large storms. It was chosen as a distribution which, when applied to a given watershed, would give the largest, or nearly largest, flood. In some cases a different distribution of rainfall might give a slightly larger peak. On the other hand, although an actual storm may equal the standard storm for the area and duration which are most critical for the watershed, past records indicate that it would fall short for other durations. This would tend to offset any increase in the flood peak due to a more adverse distribution of rainfall.

The Standard Watershed.—The standard watershed used is shown in Fig. 10. In order to determine the hydrographs resulting from the standard storm, unit hydrographs were developed for various sizes of the shed, following the empirical methods and formulas developed by F. F. Snyder, Jun. Am. Soc. C. E. (see Item (17), Appendix) for streams in the Appalachian highlands. According to Snyder, the unit graph of a watershed depends on the time lag, T_p, which is defined as the time in hours from the center of mass of the rainfall that produces runoff to the time of the peak of the flood resulting from the rain. This lag, in turn, depends upon the product of the length of the stream, L, in miles, multiplied by the length measured along the stream to a point opposite the center of area of the watershed, L_c; that is,

$$T_p = 2\,(L \times L_c)^{0.3} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

A comparison of observed lags with lags computed by this formula shows a fairly close agreement—much closer for example than the case of lag versus area of watershed, where watersheds of different shape are compared.

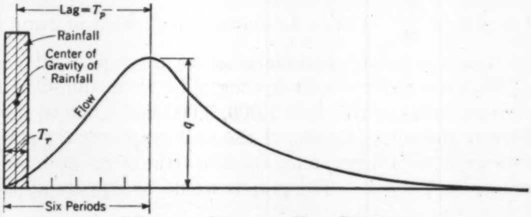


FIG. 11.—SYNTHETIC UNIT GRAPH

The period of rainfall chosen for the unit graph is one sixth the time from the start of the runoff-producing rain to the peak of the runoff. Thus the length of this period, T_r, is the lag divided by 5.5, as follows:

$$T_r = \frac{T_p}{5.5} \ldots\ldots\ldots (2)$$

This relation is made clear by reference to Fig. 11. The peak flow of the unit graph in cubic feet per second per square mile (q) is computed by the formula:

$$q = \frac{400}{T_p} \ldots\ldots\ldots\ldots (3)$$

This is the peak flow due to 1 in. of runoff in the time, T_r. The coefficients in these formulas, 2 and 400, are believed to vary for differences in slope and valley storage, and are average values which give fairly consistent results for streams in the Appalachian highlands. From the curves in Fig. 12, the percentages of total runoff occurring each day after the start of the rain can be read for any value of the lag. With these percentages, and the time and size of the peak computed by the formulas, it is possible to construct a unit graph in terms of

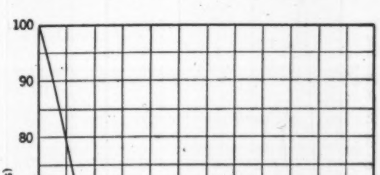


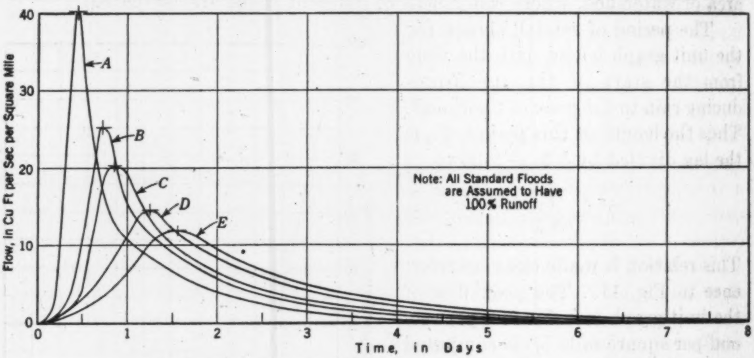
FIG. 12.—RELATION OF BASIN LAG AND DISTRIBUTION GRAPH

flow per square mile for any value of the product, $L \times L_c$. It is important to note that such a unit graph is independent of the area of the watershed, and depends only on the length of the stream and distance to the center of area.

For the standard watershed of length L (see Fig. 9) the area is the length times the average width, taken as one-fourth the length, or $\frac{L^2}{4}$. The distance to center of area L_c is $\frac{L}{2}$. Use of the average coefficients of 2 and 400 gives the standard watershed average characteristics as to slope, valley storage, etc. By means of Snyder's method unit hydrographs were computed for standard watersheds having areas of 100, 500, 1,000, 3,000 and 6,000 sq miles.

A summary of the computations of the unit graphs of the standard watershed, and the complete unit graph for the five areas of the standard sheds computed, are shown in Table 6. The graphs would be equally applicable to any watershed having the same lag.

TABLE 6.—COMPUTATION OF UNIT GRAPHS FOR STANDARD WATERSHEDS



Item	Description	Curve *A*	Curve *B*	Curve *C*	Curve *D*	Curve *E*
1	Area A, in hundreds of square miles	1	5	10	30	60
2	Length L, in miles	20	44.7	63.3	110	155
3	Maximum width b, in miles	6.36	14.2	20.1	35.0	49.3
4	Length L_c, to center of area, in miles	10	22.3	32.6	55.0	77.5
5	Product $L \times L_c$, in hundreds of square miles	2	10	20	60	120
6	Lag T_p, in hours	9.9	15.9	19.7	27.5	34
7	Time of rain T_r, in hours	1.8	2.9	3.6	5.0	6.3
8	Peak flow q from a 1-in. runoff in time T_r, in cubic feet per second per square mile	40.5	25.2	20.3	14.5	11.8

The Standard Flood.—The standard flood results from the standard storm occurring on a standard watershed. By means of the unit hydrographs previously developed, the three types of standard flood were determined for the five areas. The storms were read off at the intervals required by the unit hydrograph from the rainfall-duration curves, Figs. 7(b), 7(c), and 8(b). In the case of the small areas, with short intervals, the storm was not traced to the full 5 days, since the rainfall at the ends was so small, relatively, that it had little effect on anything but the total length of the hydrograph. The runoff coefficient was taken at 100%.

TABLE 7.—COMPUTATION OF STANDARD FLOODS

Item	Description	Curve *A*	Curve *B*	Curve *C*	Curve *D*	Curve *E*
	(*a*) WINTER FLOOD, OHIO AXIS					
1	Maximum rain in time T_r and over area *A*, in inches	4.0	3.47	3.20	2.7	2.35
	Peak Flows, in Cubic Feet per Second per Square Mile:					
2	From maximum period of rain	162	87.4	65.0	39.1	27.8
3	From entire storm	259.5	157.1	127.1	82.7	62.0
4	Ratio, Item 3 to Item 2	1.6	1.8	1.96	2.11	2.23
	(*b*) SUMMER FLOOD, OHIO AXIS					
5	Maximum rain in time T_r and over area *A*, in inches	7.2	6.8	6.2	5.0	4.3
	Peak Flows, in Cubic Feet per Second per Square Mile:					
6	From maximum period of rain	292	171	126	72.5	50.6
7	From entire storm	371.8	229.2	179.6	108.3	79.1
8	Ratio, Item 7 to Item 6	1.27	1.34	1.42	1.49	1.58
	(*c*) SUMMER FLOOD, ATLANTIC AXIS					
9	Maximum rain in time T_r and over area *A*, in inches	7.8	7.4	7.1	6.7	6.45
	Peak Flows, in Cubic Feet per Second per Square Mile:					
10	From maximum period of rain	316	186	144	97.2	76.0
11	From entire storm	435.7	269.3	215.2	149.7	122.5
12	Ratio, Item 11 to Item 10	1.37	1.45	1.50	1.54	1.61

Table 7 shows the peak flows for the five areas computed; and, for comparison, the flow resulting from the rain of the most intense period. The ratios, Items 4, 8, and 12, indicate what an important part the short-time intensity of the storm plays and why daily rainfall records, for areas of this size, will not give consistent values of peak flow. Complete hydrographs are shown in Fig. 13 for Ohio standard floods on two watershed areas.

The peak flows are plotted against drainage area in Fig. 14, for each type of standard flood. The lines in Fig. 14 are seen to be curved concave downward. This curvature is the result of the manner in which the rainfall varies as between large and small areas. Since all three lines were derived by using the same watershed, variations in the position, slope, and curvature of these lines are due solely to the size and structure of the storm.

A common method of comparing floods from areas of different size is to assume that the flow varies as the square root of the area. A line showing a flow of $\frac{5{,}000}{\sqrt{A}}$ is shown in Fig. 14 for comparison. The standard floods vary closely as the 0.3 power of the watershed area for areas between 100 and 1,000 sq miles, and the total peak flow varies as the 0.7 power of the area. From 1,000 to 10,000 sq miles, the Atlantic summer flood continues to vary at this rate, but the winter floods vary more nearly as the 0.4 power, and the Ohio summer flood as the 0.45 power of the watershed area. These differences are the result of differences in the structure of the storm on areas greater than 1,000 sq miles. The more nearly uniform the storm in both duration and area, the less will be the effect of watershed area on the peak flow per square mile.

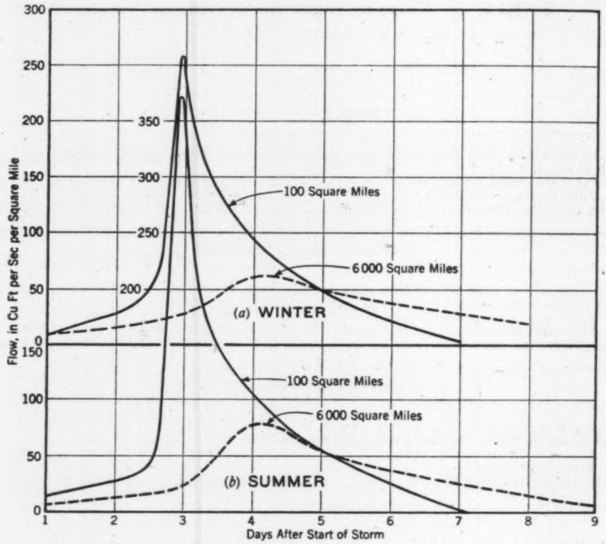


Fig. 13.—Hydrographs of Standard Ohio Floods

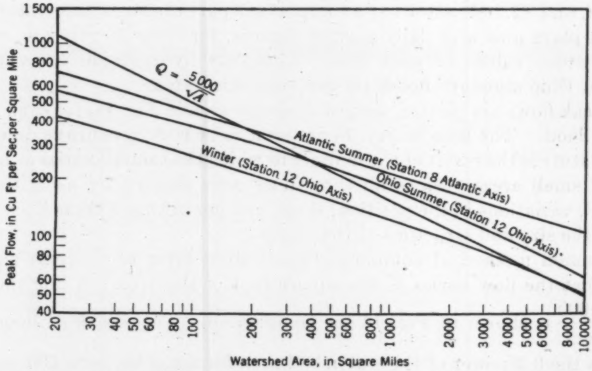


Fig. 14.—Standard Flood Peaks for Various Areas

It is evident that no single rate of variation can be applied to all seasons and areas, except as a very rough approximation. A check of the slope of the Atlantic standard flood line on Fig. 14 was made against a group of floods whose frequency has been estimated. A list of twenty-seven 1% chance floods was given by the late Allen Hazen, M. Am. Soc. C. E. (Item (19), Appendix), for watersheds located mostly on the Atlantic Coastal Plain. They were estimated by probability methods from stream-flow records, ranging from 20 to 84 years long. These floods were corrected for shape and location of watershed to make them comparable with Atlantic summer standard floods, by a method to be presently explained. An average line through the floods cited by Hazen is roughly parallel to the standard flood line, indicating the same variation of flow with watershed area. The Atlantic standard flood, with 80% runoff, is roughly 2.5 times as large as these corrected 100-yr floods. It is about twice as large as the 1,000-yr floods as computed by Fuller's formula,

$$Q(\max) = C\,A^{0.8}\,(1 + 0.8 \log T)\left(1 + \frac{2}{A^{0.3}}\right) \qquad (4)$$

using a coefficient C of 80 and time, T, of 1,000 years (Item (20), Appendix). In this case, too, the lines are roughly parallel. These comparisons indicate that the standard floods have about the same frequency for all sizes of drainage area. It also confirms the extreme rarity of floods of this size on any given watershed.

The foregoing discussion relates to the peak flow of the standard floods. The volume of the standard flood, in inches on the watershed for all water in excess of a given flow, expressed as a percentage of the peak flow, was computed from complete hydrographs of winter and Ohio summer storms, and is shown in Table 8. Except for the total storm, the watershed area has little effect on these volumes. The higher peaks, in flow per square mile, of the

TABLE 8.—VOLUME OF STANDARD OHIO FLOODS ABOVE VARIOUS RATES OF FLOW

Percentage of peak flow[a]	100-SQ-MILE AREA			3,000-SQ-MILE AREA			6,000-SQ-MILE AREA		
	Flow[b]	Volume, in inches	Percentage of total volume	Flow[b]	Volume, in inches	Percentage of total volume	Flow[b]	Volume, in inches	Percentage of total volume
(a) WINTER									
0	0	13.2	100	0	9.6	100	0	8.1	100
10	26	8.75	66.4	8.3	9.04	94.0	6.2	7.65	94.5
25	65	4.68	35.4	20.7	4.96	51.6	16.6	4.9	60.5
50	130	1.56	11.8	41.3	2.11	22	31.1	2.03	25.1
100	260	0	0	82.6	0	0	62.2	0	0
(b) SUMMER									
0	0	16.0	100	0	10.0	100	0	8.6	100
10	37	9.4	58.8	11	8.05	80.5	8	7.2	84.0
25	93	5.0	31.3	27	4.98	31.9	20	4.6	53.5
50	186	1.8	11.2	54	2.01	20.1	39.5	1.87	21.8
100	372	0	0	108	0	0	79	0	0

[a] Volume above zero flow equals 5-day rainfall. [b] Cubic feet per second per square mile.

smaller areas is just about offset by the longer time during which a given percentage of the peak is exceeded by the floods from the larger areas. The data indicate the storage needed, assuming perfect reservoir operation, to reduce the standard flood peak by various amounts.

THE MAXIMUM PROBABLE FLOOD

The maximum probable flood for a particular watershed is assumed to equal the flood caused by the maximum probable storm for that location, occurring so the area of maximum rainfall coincides with the watershed boundaries, and with a runoff coefficient normal for the season and quantity of rain. The severe condition of a perfect fit is assumed to offset the possibility of a larger storm, or of a higher than normal runoff coefficient, due to previous rain. It is not considered in the least probable that all of these factors would be present simultaneously. Increase in flow due to dam failures, or increase in flood heights due to ice jams or debris dams, is not considered.

Where a unit hydrograph of the watershed is available, the maximum probable flood can be computed from the standard storm, corrected for location. For other cases, and where only the peak flow is desired, it may be found by applying suitable corrections to the standard flood. Three correction factors are considered: One is for the effect of location, one for the effect of shape, and the third is the appropriate runoff coefficient. Although it would be desirable, no means were found of introducing the effect of slope and valley storage on the flood-producing capacity of a watershed. The extent to which these factors may influence the size of floods in the state may be inferred from the variations Snyder found in the two constants of his formulas. The combined effect of this variation for extreme cases is believed to be about 15% either way.

Location Correction.—The correction factor for location can be read from Fig. 6. It was assumed that the maximum probable storm varied according to the slope of the storm profiles on which this map was based; it follows that the floods resulting from this storm on a standard watershed will vary the same way. The Juniata Basin, above Newport, Pa., is shown in Fig. 15 as an example. From Fig. 6, the summer correction is read as 115%. The maximum probable flood at this point, therefore, would be 15% greater than the standard flood, due to the difference in location.

Shape Correction.—During the maximum probable flood an actual watershed will have the same rainfall as a standard watershed of the same area. It will have the same unit graph (in flow per square mile) as some other standard watershed having the same product ($L \times L_c$) and therefore the same lag. If the peak flow of the standard flood from this second watershed is corrected for the difference in the quantity of the rainfall, due to the difference in area, it will represent the maximum probable flood, at 100% runoff, from the actual watershed—subject, of course, to the location correction. The variation of rainfall with area is slightly different for different durations, but if a duration equal to the time of rain of the unit hydrograph is used, any error from this cause will be slight. The shape correction factor is defined as the ratio of the maximum probable flood to the standard flood for an area equal to the area of the actual watershed.

Referring to the example of the Juniata Basin, in Fig. 15, the value of $L \times L_c$ is 72 × 137, equaling 9,864. The standard watershed with this value will have an area of $\frac{9,864}{2} = 4,932$ sq miles. It is indicated in Fig. 15 by the larger ellipse. The standard Ohio summer flood from this area is 89 cu ft per sec per sq mile. The time of rain is about 6 hr, and the 6-hr rain on 4,932 sq miles is 4.5 in., and on 3,354 sq miles (the area of the Juniata) is 5 in. Thus, the Juniata flow will be $\frac{5.0}{4.5}$, or 111% of the flow from the standard shed with the same product, $L \times L_c$. This equals 111% of 89, which is 99 cu ft per sec per sq mile. The standard flood from a standard watershed, with an area equal to the Juniata (indicated by the smaller ellipse), is shown in Fig. 14 as 106 cu ft per sec per sq mile. The Juniata flood is thus $\frac{99}{106}$, equaling 93% of the standard flood, which is the desired shape-correction factor.

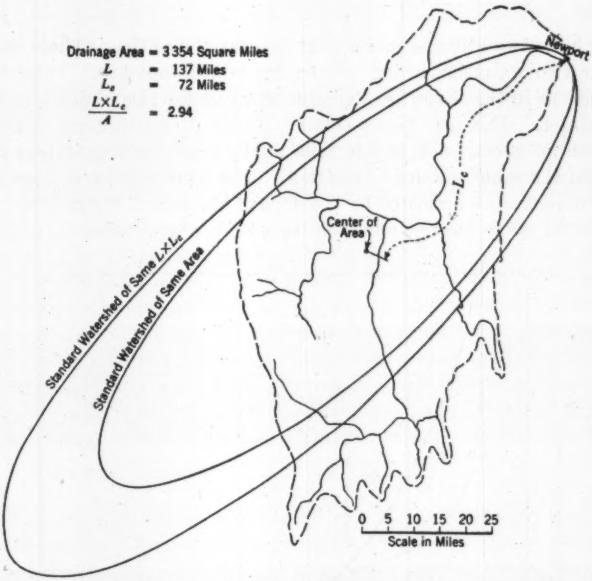

Drainage Area = 3354 Square Miles
L = 137 Miles
L_c = 72 Miles
$\frac{L \times L_c}{A}$ = 2.94
Newport
L_c
Center of Area
Standard Watershed of Same $L \times L_c$
Standard Watershed of Same Area
0 5 10 15 20 25
Scale in Miles

FIG. 15.—COMPARISON OF ACTUAL AND STANDARD WATERSHED; JUNIATA RIVER AT NEWPORT, PA.

The variation in shape of a watershed from the standard can be measured by the ratio, $\frac{L \times L_c}{A}$. For the standard watershed, this shape ratio is 2. The shape-correction factor for a series of areas with shape ratios from 0.5 to 5 was computed, as explained, for each of the three storm types, and for various areas. The results are listed in Table 9. The range covers the variation in natural watersheds and corresponds, roughly, to a range of length-to-average-width ratios of 1 to 10. An area whose width is greater than the length will

TABLE 9.—SHAPE-CORRECTION FACTORS

Shape ratio	(a) OHIO WINTER STORMS				(b) OHIO SUMMER STORMS				(c) ATLANTIC SUMMER STORMS			
	Drainage Areas, in Square Miles:											
	100	1,000	6,000	Average	100	1,000	6,000	Average	100	1,000	6,000	Average
0.5	1.27	1.16	1.15	1.19	1.35	1.33	1.29	1.32	1.40	1.32	1.29	1.34
1.0	1.15	1.06	1.06	1.09	1.18	1.15	1.12	1.15	1.19	1.14	1.13	1.15
2.0	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
3.0	0.94	0.97	1.00	0.97	0.92	0.94	0.94	0.935	0.93	0.925	0.935	0.93
4.0	0.89	0.94	0.94	0.92	0.875	0.895	0.875	0.885	0.88	0.885	0.895	0.89
5.0	0.825	0.92	0.905	0.883	0.83	0.86	0.85	0.85	0.825	0.845	0.860	0.84

generally have two streams meeting at or near the outlet. These should be treated as two watersheds, and the results added together. Table 9 shows that variations in the shape have greater effect on the size of floods in summer than in winter. This is to be expected. Under a long, uniform storm, shape would have no effect, each square mile finally reaching a condition where it contributed the same amount to the flow. The winter storm is more uniform than the summer, as is indicated by the slope of the lines on the rainfall-duration curves; hence, shape has less effect on the size of winter floods.

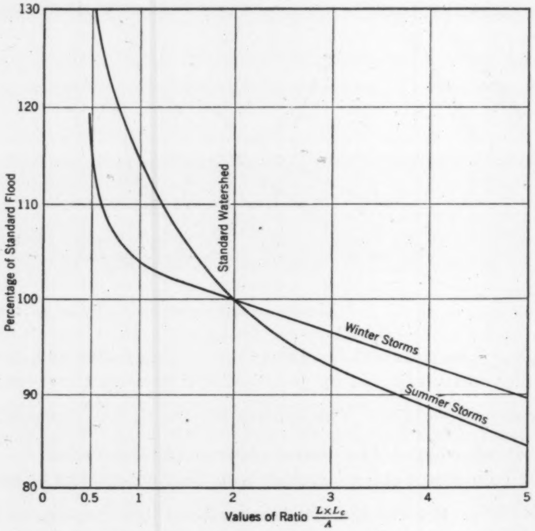


FIG. 16.—EFFECT OF WATERSHED SHAPE ON FLOODS PRODUCED BY STANDARD STORMS

Correction factors for the two types of summer storms proved to be so nearly alike that one set of values may be used for both. In Fig. 16, the average shape-correction factor for summer and winter storms, taken from

Table 9, is plotted against values of the shape ratio, $\frac{L \times L_c}{A}$. These curves can be used to find the shape-correction factor after the length of stream and length to the center of the area have been found. For this procedure, only a map of the watershed is needed. If the actual lag of the watershed can be determined from field observations on a few floods, the accuracy of the estimate will be improved. In this case, the flow per square mile will be the same as from a standard shed with the same lag, corrected for difference in rainfall on the two areas.

Runoff Coefficient.—The runoff coefficient is the third and last factor to apply to the standard flood to estimate the maximum probable flood from a particular watershed. As used herein, the runoff coefficient may be defined as the total surface runoff due to a particular storm, expressed as a percentage of the rainfall causing it. The runoff coefficient is not a constant, but varies with the season of the year, the size of the storm, and the character of the watershed.

Seasonal analysis of the runoff coefficient was attempted by plotting all the runoff coefficients (see Items (7), (8), (9), and (10), Appendix) against the time of year when the flood occurred. A line defining the upper limits of the scattered points was drawn. Its shape strongly suggested the mean annual temperature curve reversed, but its position was about one month later. The highest values of the runoff coefficient thus appeared to vary inversely with the temperature of the month preceding the flood. This is probably due to the direct effect of temperature on the ground-water levels. It was decided, therefore, to use the average monthly temperature of the preceding month as a numerical expression of the season of the year. This facilitated comparison of runoff coefficients from widely separated points where the conditions in a particular month might be quite different. The runoff coefficients were classified into groups according to the mean temperature of the month preceding the flood, the first group running from 10° to 30° F, the others by 10° steps to 80° F. They were then plotted against the total inches of rain causing the flood. Two samples of these plottings are shown in Fig. 17. Lines connecting the maximum and minimum values, and smooth curves to represent the average maximum, were drawn. Points above 100%, due to snow runoff, were neglected as there was no way to separate this from the rain runoff. Although still widely scattered, the points reveal the trend of higher coefficients with lower temperatures and heavier rainfalls. The average maximum curves are shown together in Fig. 17(*c*).

The other causes for variation in the runoff coefficient, which account for the range in values for the same rainfall and temperature, are mostly associated with the particular watershed and may be assumed as constant for that shed. They include such things as the nature of the soil, kind and amount of vegetation, steepness of the slopes, presence of swamps, and the like. In general, the variations either way from the average, due to these causes, are less than the range due to season and size of rain.

Some allowance may be made for the peculiar character of the watershed if the coefficients for a few floods are available. Such values may be plotted on

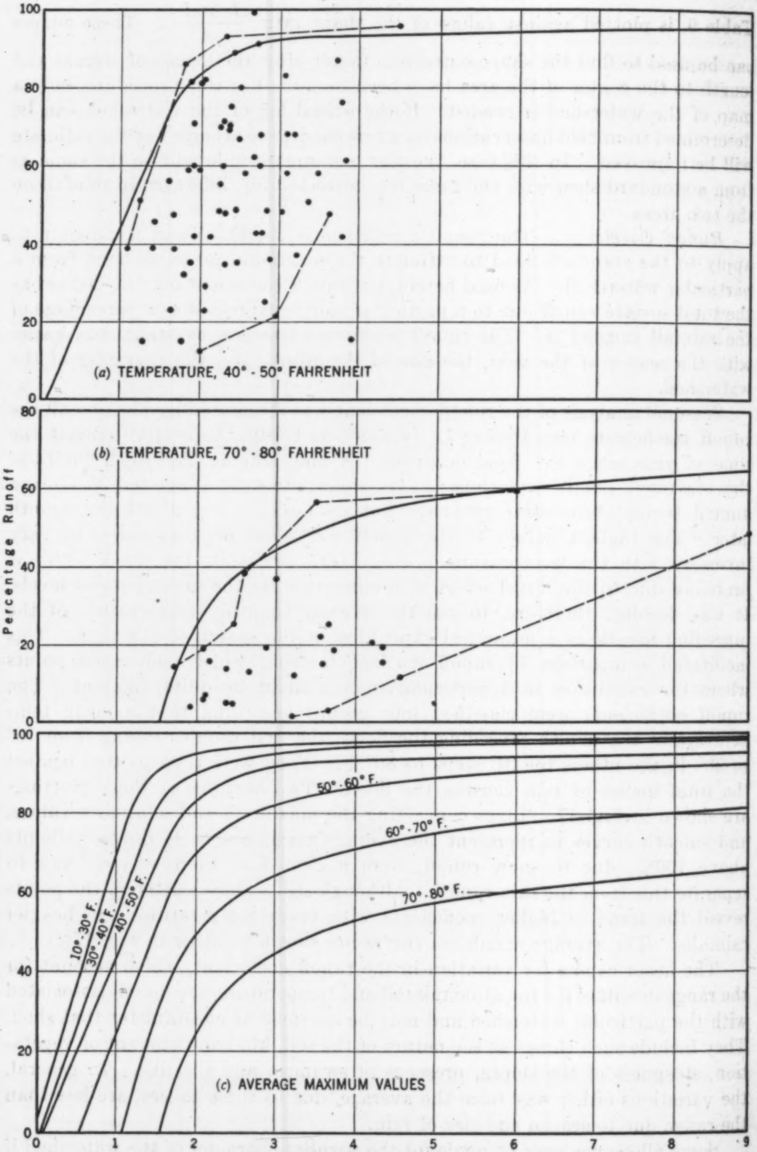


FIG. 17.—VARIATION OF THE RUNOFF COEFFICIENT WITH TOTAL RAINFALL AND WITH THE TEMPERATURE OF THE MONTH PRECEDING THE FLOOD

the figures for the appropriate amount of rain and the temperature of the preceding month. By observing the position of these points with relation to the maximum and minimum lines, the probable values of the coefficient for different seasons and larger amounts of rain may be estimated. For winter storms, the possibility of snow runoff must be considered and an allowance made for it.

In selecting a coefficient for the maximum probable storm, precision such as is required for estimating or predicting an actual flood from rainfall data at hand is meaningless, since all the conditions surrounding the future storm must be assumed. Neither is it necessary to use the highest coefficient on the record except as its size may be affected by the size of the storm. Aside from this, the storm is as likely to occur when the coefficient is low, due to a warm preceding month, as when it is high. The combination of the maximum probable storm with an extreme value of the coefficient is considered improbable. In the absence of any specific data, a value of the coefficient of 80% to 90% for summer and 120% for winter would seem to be safe for the purpose.

Estimating the Maximum Probable Flood.—The process of determining the maximum probable flood may be summarized thus:

$$\text{Maximum probable flood} = \text{standard flood} \times \text{location correction} \times \text{shape correction} \times \text{runoff coefficient} \times \text{drainage area} \qquad (5)$$

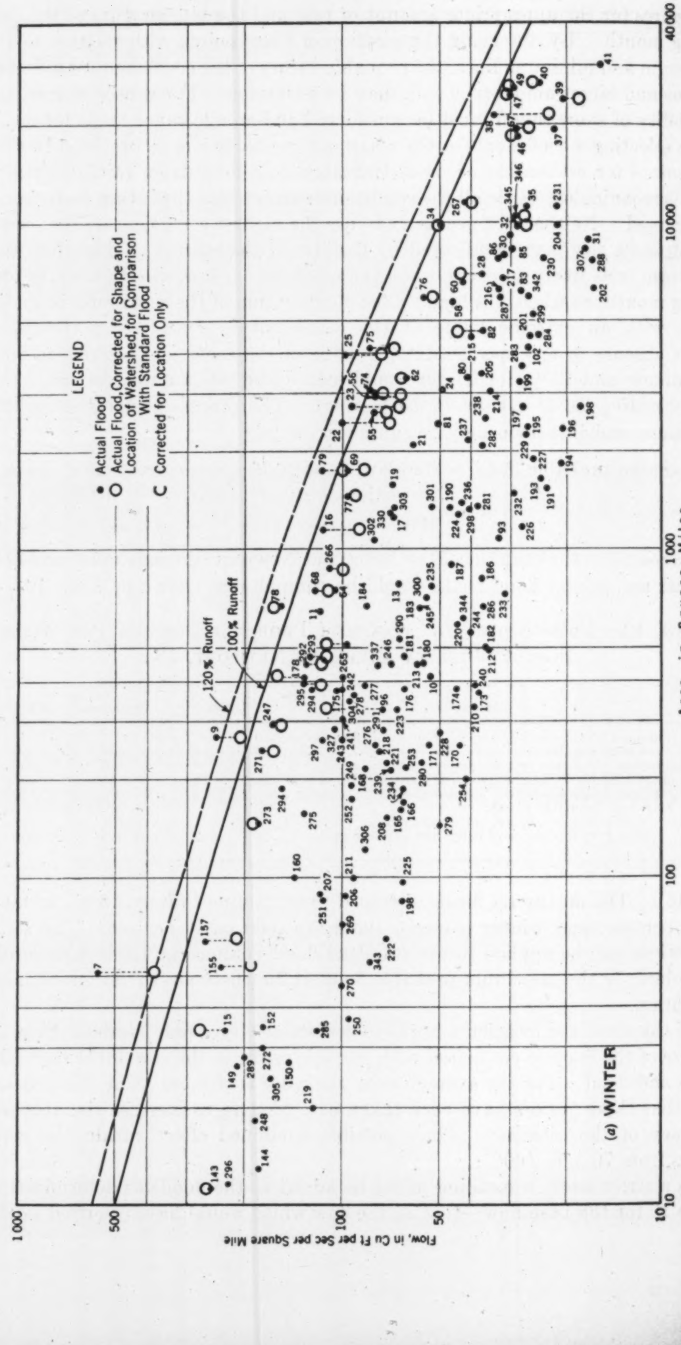
To return to the example of the Juniata at Newport, the maximum probable summer and winter flood peaks would be computed as shown in Table 10 (see Fig. 15). The maximum floods of record were: Summer (June, 1889), 240,000 cu ft per sec; and winter (March, 1936), 215,000 cu ft per sec. The same corrections can be applied to the standard flood volumes in Table 8 to obtain the volume of the maximum probable flood at Newport above various channel capacities.

TABLE 10.—COMPUTATION OF MAXIMUM PROBABLE SUMMER AND WINTER FLOODS; JUNIATA RIVER AT NEWPORT, PA.

Item	Description	See Fig.	Summer	Winter
1	Standard flood from 3,354 sq miles, in cubic feet per second per square mile	14	106	80
2	Location correction (percentages)	6	115	98
3	Shape correction (percentages) for a shape ratio of 2.94	16	93	97
4	Assumed runoff coefficient (percentages)	....	90	120
	Maximum Probable Flood:			
5	Cubic feet per second per square mile	....	102	91
6	Total, in thousands of cubic feet per second	....	340	306

In this case, the location and shape corrections have only a minor effect on the size of the flood as compared with possible errors in the size of the standard storm and flood. For the average case, their use is of more value as an assurance that these factors have been taken care of than to increase the absolute accuracy of the estimate. Their possible combined effect within the state ranges from 70% to 160%.

In a strict sense, something might be added to the floods as determined in Table 10 for the base flow—that is, the flow which would have occurred in the



(a) WINTER

LEGEND
● Actual Flood
○ Actual Flood, Corrected for Shape and Location of Watershed, for Comparison With Standard Flood
C Corrected for Location Only
120% Runoff
100% Runoff
Flow, in Cu Ft per Sec per Square Mile
1000
500
100
50
10
Area, in Square Miles
10
100
1000
10000
40000

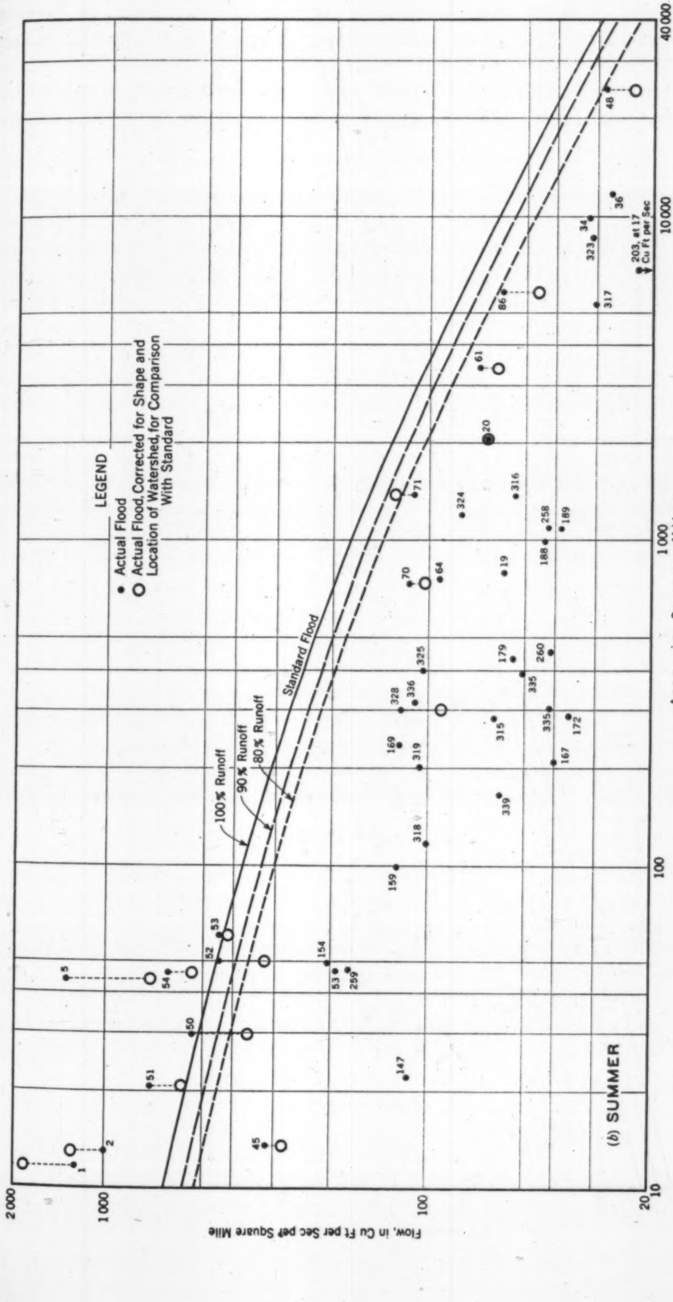


FIG. 18.—COMPARISON OF STANDARD AND ACTUAL FLOODS; STATION 12, OHIO AXIS

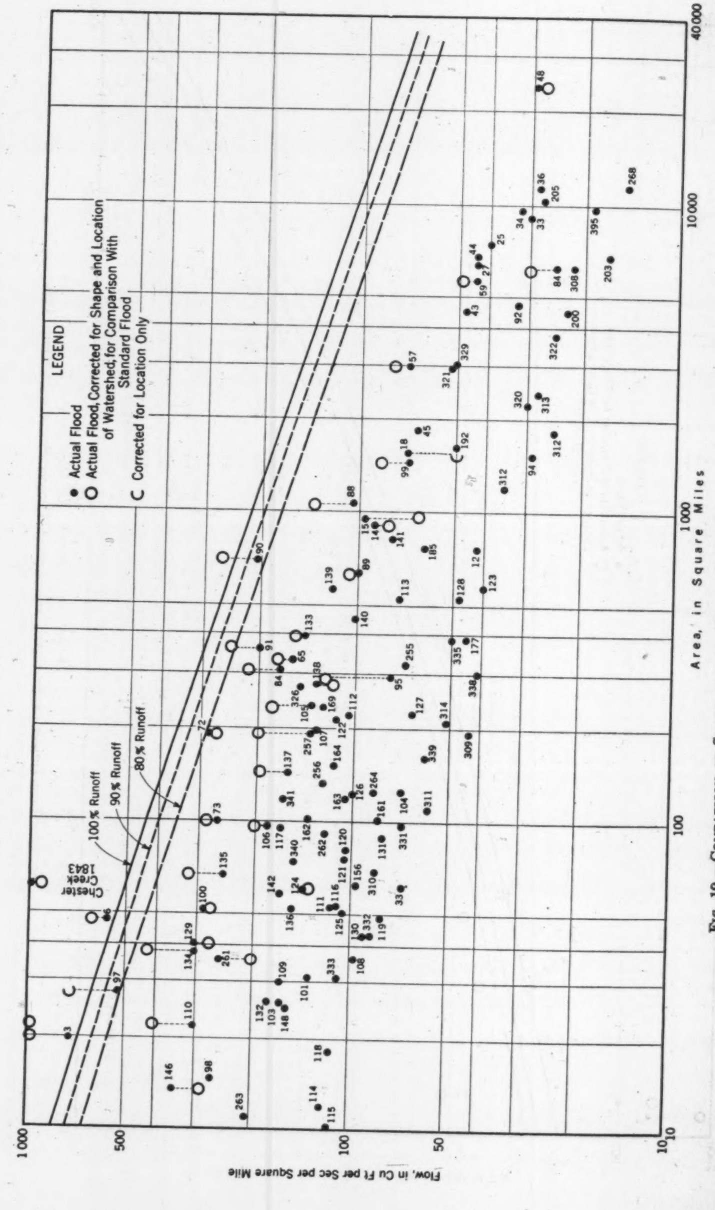


FIG. 19.—COMPARISON OF STANDARD AND ACTUAL FLOODS; STATION 8, ATLANTIC AXIS; SUMMER

stream had there been no rain. In comparison with floods as large as the maximum probable flood, any normal base flood would be insignificant. Occasionally, a flood occurs superposed on a large base flow from a preceding storm, but such an event is considered to be offset by the unlikelihood of its coinciding with the maximum value of the other factors that affect flood size.

COMPARISON WITH FLOOD RECORDS

By applying the three correction factors to an actual recorded flood peak, an estimate can be made of the flood that would have been produced by the same storm had it occurred on a standard shed at station 12 or 8. A comparison of this corrected actual flood with the standard flood for the same drainage area will give a measure of the relative size of the storm, and of the flood resulting from it. A list of all the large floods in Pennsylvania and the adjacent states was compiled (see Appendix, Items (13) to (16)) and the floods separated into the three types—winter, Atlantic summer, and Ohio summer. In Figs. 18 and 19 these floods are shown as peak flow per square mile against watershed area. The three standard floods were drawn on the figures for comparison, together with lines representing 80% and 90% for summer runoff and 120% for winter runoff. Those floods that exceeded the standard flood, or approached it closely, were then corrected for location and shape of watershed to make them comparable with the standard floods. For a few floods, as indicated, maps could not be secured to make the shape correction. The runoff coefficient is unknown for most of them, and no correction was made for it. Figs. 18 and 19 show the extent to which the maximum probable flood, as derived from storm data, checks the past record. There is a tendency for the standard flood to be exceeded on the small areas (less than 100 sq miles) to about the same extent as a few short-duration rainfalls exceeded the standard storm (see Figs. 7(*a*) and 8(*a*)).

For the larger areas the Atlantic standard flood (Fig. 19) exceeds the actual floods by a considerable margin. Considering only the flood points, it appears that a steeper line, perhaps one represented by the formula

$$Q = \frac{K}{\sqrt{A}} \qquad (6)$$

would fit the data better. Because of the nature of the data plotted, however, it is believed that such a curve would not represent floods having the same probability of occurrence as between large and small areas. The record for the Atlantic coastal region includes very few basins of 10,000 sq miles, or more, in area. At the other end of the scale, there are hundreds of watersheds of 50 to 100 sq miles in area. Although all of these are not gaged, when one of them produces a notable flood some estimate is generally made of it and it finds its way into the records. Thus, the Chester Creek flood of 1843, which has not been approached anywhere in this region in the succeeding hundred years, is included and would probably be included had it occurred on any one of a hundred streams of comparable size. This flood obviously represents a very different degree of probability in respect to occurrence on a particular designated watershed than do the recorded floods for areas around 10,000 sq miles.

The Ohio standard floods, where the actual flood records for large areas are more numerous, are in better agreement.

Both the standard flood lines, derived entirely from rainfall data, and the records of actual floods, as corrected, appear to indicate an upper limit to flood size which will rarely be exceeded.

CONCLUSION

It is not the writer's intention to present another flood formula or curve of maximum floods. Neither is the maximum probable flood offered as a criterion to be used blindly for design of flood-control projects or spillways, since factors of cost, benefits from protection, and damage done by failure are all involved in such decisions. The methods are described and the data displayed with the hope of making clearer the flood-producing mechanism; of giving quantitative significance, however approximate, to the relative importance of the factors that influence the size of floods; and finally, to serve as a guide to judgment in selecting the size of flood to be provided for.

ACKNOWLEDGMENT

Most of the work on which this paper is based was done for the Pennsylvania Flood Control Bureau. The writer wishes to acknowledge the help of G. Gale Dixon, M. Am. Soc. C. E., former chief of the Bureau, for valuable suggestions and criticisms; of Mr. Snyder for advice concerning the application of his synthetic unit graphs; and of C. L. Rice and the late F. M. Felker, Jun. Am. Soc. C. E., for assisting with the computations involved.

APPENDIX

SOURCE AND SCOPE OF DATA

(1) The data on the storm paths were taken from U. S. Weather Bureau records, in which maps are available showing the path of the low-pressure areas since about 1890. Synoptic weather maps, which would throw considerably more light on storm behavior, are a development of recent years, and would not supply a sufficiently long record.

(2) The data on storms used on the storm profiles come mainly from "Storm Rainfall in Eastern United States," published in 1936 by the Miami Conservancy District. This report covered all the storms in the United States east of the 104th meridian which had a rainfall of 6 in. in 3 days at five or more rainfall stations for the period 1892 to 1933. The total number was 280 storms, and included 3 before 1892. Of these, the time-area-depth curves for the 73 most important storms are given in the book. Two more storms, Nos. 4 and 265, for which such curves were not given, but which occurred near Pennsylvania, were estimated and included. The following four other storms occurring in or near the state between 1933 and 1937 were added: July 7–8, 1935, in New York; September 4–5, 1935, in Maryland; March, 1936, in Pennsylvania and New England; and January, 1937, in Tennessee and Kentucky. No attempt was made to bring the report up to date for the entire

United States; but with these additions, the data include the largest storms in and near the state for the 45 years 1892–1937.

(3) The "Miami Report" was also the source of data on the highest rainfall at a single station in each 2° quadrangle, and of the 1-day to 5-day rainfall having a 100-yr frequency at stations 8 and 12.

(4) For shorter durations, data were taken from "Rainfall-Intensity-Frequency Data," by David L. Yarnell, *Miscellaneous Publication No. 204*, U. S. Dept. of Agriculture.

(5) Other data on short-duration rainfall were taken from a report made by Arthur E. Morgan, M. Am. Soc. C. E., to the Pennsylvania Water Supply Commission, in July, 1921. This contained some very old records—notably, Catskill, N. Y., in 1817, and Chester, Pa., in 1843. Some of the recent hourly rainfall records were taken from U. S. Weather Bureau records.

(6) Data on the runoff coefficient were secured from the following sources:

(7) The *Journal* of the Boston Society of Civil Engineers for September, 1930, gives runoff coefficients for the Ware, Swift, Westfield, Nepaug, Farmington, and Waschuset rivers in New England and the Miami River in Ohio. A total of about fifty values for watersheds, ranging from 50 to 2,500 sq miles, are given.

(8) *Water Supply Paper No. 772*, U. S. Geological Survey, contains runoff coefficients for the Delaware, Muskingum, Susquehanna, and Wabash rivers. A total of twenty-five values is given for areas ranging from 3,000 to 8,000 sq miles.

(9) The U. S. Corps of Engineers developed runoff coefficients for streams in and near the Upper Susquehanna Basin, the Chemung, Cohocton, Unadilla, and Chenango rivers. Some 200 values were used from this list. Unlike the remainder of the data, the base flow was not excluded in computing the coefficient. For this reason, values for the smaller rains, where the base flow might have been a large part of the total, were excluded and nearly all the records used were for rains of 2 in. or more (unpublished).

(10) About fifteen values of the coefficient, computed by the writer for various streams in the state, were also used (unpublished).

(11) All the data represent the runoff coefficient for one particular flood, based on the rain that caused it. Rainfalls of various intensities and durations are naturally included.

(12) The flood records plotted in Figs. 18 and 19 were taken from "Flood Flow Characteristics," by C. S. Jarvis, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 985, *et seq.* This contains a very complete list of floods up to 1926. It was supplemented and brought up to date with data from the following:

(13) *Journal*, Boston Soc. of Engrs., September, 1930.

(14) "New York State Flood of 1935," *Water Supply Paper No. 773E*, U. S. Geological Survey.

(15) "The Floods of March 1936," *Water Supply Papers Nos. 798, 799 and 800*, U. S. Geological Survey.

(16) The Water Supply Papers, in addition to the particular flood discussed, give values for the highest previous flood and thus serve to bring Mr. Jarvis' comprehensive record up to date. As far as possible, these data were compared and checked with a list of the largest floods in Pennsylvania, compiled by J. W. Mangan, M. Am. Soc. C. E., of the Pennsylvania Division of Hydrography, in which a number of the previously accepted values for peak flow have been corrected.

Other sources cited are:

(17) *Transactions*, Am. Geophysical Union, Part 1, 1938, pp. 447–454.
(18) *Proceedings*, Am. Soc. C. E., June, 1938, p. 1296.
(19) "Flood Flows," by Allen Hazen, John Wiley and Sons, Inc., New York, N. Y., 1930, pp. 63 and 64.
(20) *Transactions*, Am. Soc. C. E., Vol. LXXVII, December, 1914, p. 564.

DISCUSSION

Joseph L. Benson,[2] **Assoc. M. Am. Soc. C. E.** (by letter).—By his thorough but concise presentation of the factors and principles involved in the determination of the maximum probable flood, Mr. Ruff has made a distinct and valuable contribution to the field of hydraulic engineering. Experienced engineers will recognize not only the merits of the data presented but will also appreciate the author's recommendation for caution in accepting his formulas, tables, and graphs as the missing pieces of their own specific jig-saw problems. The critical engineer may not accept the author's findings entirely, but he will undoubtedly find, among the research data presented, certain tools that can be utilized in his own work. It appears obvious that the author does not intend for engineers to dissect his paper for rules-of-thumb to be used as a substitute for whatever rule-of-thumb they may have used in the past for solving their own particular problem.

The writer believes additional thought on this subject may be provoked by calling attention to the following comments:

1. Mr. Ruff describes his "maximum probable flood" as one "so large that the chance of its being exceeded is no greater than the hazards normal to all of man's activities." This is a concept rather than a definition. What are the hazards normal to all of man's activities? Utility will be added to the value of the paper if the author, together with other eminent engineers, can furnish examples for which his "maximum probable flood" would be the design flood and, for the benefit of those who work with frequencies, also correlate this flood to frequency of occurrence based on presently available data. What are the conditions or circumstances for which the author's maximum probable flood would be the design flood? This is important because too often research reports have failed to reach their objective or ultimate possibilities, due to the omission of qualifying or related statements. The writer believes that the maximum probable flood derived from the author's methods and data would be equivalent to the design flood for a spillway structure (either dam, highway, railroad, etc.), failure of which might cause loss of life, suspension or dislocation of business activity, and in addition direct dollar damages of several times the cost of the entire project. Frequency values of this "maximum probable flood" can be determined for certain particular localities by utilizing the flood-frequency curves of those localities. The "maximum probable flood" will be a computed discharge for a particular locality. This computed discharge has a frequency value as determined from the flood-frequency curve. The approximate frequency of the enveloping curves could be ascertained by the foregoing determination for these points which control the location of the enveloping curves.

2. If sufficient agreement could be obtained on the occasion and purpose for which the maximum probable flood would be the design flood, it would be possible to establish a series of relationships between the protection offered by, and the magnitude of, this design (maximum probable) flood and the protection

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[2] Care Federal Power Comm., Baltimore, Md.

required by, and the magnitude of design floods for, other situations. At a specific location, the design flood for a bridge on a "farm-to-market" dirt road would be different from a large dam or important transcontinental railway bridge. It is obvious, therefore, that the structure itself affects the magnitude of the design flood. It is also believed that sufficient importance has not been given to the effect of the possible downstream damages on the design flood used for spillway structures and flood storage. Most engineers have realized, in a qualitative sense, that less protection is needed where resulting damages would be small if failure should result. From a quantitative standpoint, engineers have found very little in print on this subject and have had to depend on their own judgment. The effect of structure and potential downstream damage on the magnitude of the design flood is appreciable.[3]

3. The author's method for determining the maximum probable flood involves rotation of the axis of the storm, when transposed over the particular drainage area, to give the maximum runoff from the particular storm. Mr. Ruff assumes that the "severe condition of a perfect fit" compensates for the vagaries of nature in causing a larger storm or the effect of a higher than normal runoff coefficient due to a previous storm. The writer believes the compensation should be provided for by means other than rotation of the axis of the storm. Although little data have been published concerning either the propriety or impropriety of rotating the axis of a storm, it appears best to transpose the storm without rotation.[4, 5] The writer believes that the more uniform the pattern of the storm paths, the less basis can be found for rotation of the axis of the storm. Additional data on the meteorology of floods appear necessary and may be available.[6]

4. The validity of transposing the isohyetals of a storm to determine the "maximum probable flood" depends on: (*a*) The probability that the storm would or could center over the drainage area in the future, and (*b*) reliability of the isohyetals drawn from the recorded rainfall. In the former case engineers have been sufficiently warned concerning the application of air-mass analysis without expert advice.[6] With regard to the latter, there is nothing so spotty as spotty rainfall.[7] There should be a sufficient number of Weather Bureau stations within the area of heavy rainfall not only to delineate the isohyetals properly, but also to permit sufficient reliance to be placed on them. If there is a choice between using the method of transposing isohyetals or using flood runoff data, the writer suggests using both. Of course, the weight to be given to each solution will depend on the reliability of the data used for each method, especially the period of time for which the rainfall and river runoff data are available.

5. The author uses the enveloping curve for the storm rainfall producing the maximum probable flood and provides no factor of safety as a surcharge

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[3] "Recreational Use of Forest Waters," by Clifford Allen Betts, M. Am. Soc. C. E., *Engineering News-Record*, August 29, 1940, Fig. 4, p. 61.

[4] "Measuring the Discharge of Texas Streams," by J. L. Lochridge, M. Am. Soc. C. E., *Civil Engineering*, September, 1937, p. 638.

[5] "The Sardis Dam and Reservoir," by Norman R. Moore, M. Am. Soc. C. E., *loc. cit.*, June, 1939, p. 349.

[6] "Flood-Protection Data," Progress Report of the Committee, *Proceedings*, Am. Soc. C. E., February, 1938, p. 338.

[7] *Proceedings*, Am. Soc. C. E., January, 1940, p. 178.

(at points controlling the location of the enveloping curve) for possible storms in excess of those already recorded. By rotation of the axis of the storm as indicated in the preceding paragraph, the author actually obtains a factor of safety, in the sense that rotation of the axis of a storm will produce a flood several times larger than would be the case without rotation. However, would not the experiences with enveloping curves of flood runoff be repeated with enveloping curves of maximum rainfall? William P. Creager, M. Am. Soc. C. E., believes that if enveloping curves of record storms "* * * were derived and plotted, they would show the same increase with time, that is, an increase with the accumulation of data."[8] It is possible that the author can offer additional information on this point. The enveloping curves of maximum rainfall may be a definite derivative of the pattern of storm intensities, but, when the location and slope of the enveloping curve are determined by only two or three storms, there is a strong possibility that such curve will be pierced at several points as time goes on. This possibility need not detract from the value of the author's work, if it is definitely understood that the enveloping curve is a tool in determining the maximum probable flood and is not of itself a representation of the maximum probable storm.

6. Mr. Ruff's definition of runoff coefficient is admirably suited to his method of determining the probable maximum flood. However, more accuracy can be obtained by converting fallen snow to equivalent inches of rainfall. If this change were adopted, the author's method could still be utilized by adjusting the enveloping curve of the winter storms to include the expected snowfall conditions. Many engineers will find great interest in the emphasis on seasonality and temperature, size of storm, and character of the watershed as affecting the runoff coefficient. Apparently, size of drainage area does not affect the runoff coefficient other than to provide a place for the watershed characteristics to take effect.

7. Engineers who have made flood-flow studies based on the same storms used by the author should present their findings. For example, the storm designated as 244, March 12–15, 1929, near Elba, Ala., was used as the basis for determining the spillway capacity of the Sardis Dam in Mississippi. Likewise, the storm designated as 248 may have been used in spillway studies of Saluda Dam in South Carolina. It is logical to assume that, whenever important hydraulic structures have been erected in the locality, adjacent and subsequent to one of the designated storms, the designing engineer would compare his design data with those obtained by using the author's methods. Engineers who have used one or more of the specific storms represented in Tables 2 and 3 could, therefore, contribute comparative data to the discussion.

8. The value inherent in some parts of the author's presentation cannot be overemphasized. So much emphasis has been placed on peak flows that some engineers talk of nothing else, whereas equal or greater emphasis should be placed on flood volume or flood volume above a specified rate of flow. Fig. 14 and Table 8 illustrate both peak flow and flood volume. Peak flows have utility as: (*a*) An index of flood magnitude and hydrograph construction; (*b*)

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[8] "Possible and Probable Future Floods," by William P. Creager, *Civil Engineering*, November, 1939, pp. 668–670.

a criterion for spillway design of run-of-river hydro-plants or other structures where no flood storage is used in the design; and (*c*) development of backwater curves. Since most reservoirs provide an appreciable flood-storage capacity (usually that storage between the maximum and normal water levels), the importance of flood volume cannot be overemphasized. In the case of most flood-control reservoirs, it is flood volume above a stated rate of outflow, usually limited to non-overflow or nominal overflow of the river channel. Where downstream conditions do not limit the outflow for structures, it is flood volume above the average spillway capacity. It is realized that practically all engineers take flood volume into consideration when that item affects the design of the spillway structure; but further emphasis on flood volume above the relevant flow is desired in technical publications and discussions. The relative importance of peak flow and flood volume, of course, is dependent on the locality and the design of the structure. The difference between inflow and outflow is obviously the difference in the volume of water going into or out of storage. One extreme is the run-of-river hydro-plant with little or no storage where outflow or spillway capacity must approach or equal the inflow, and the other common extreme is the normally empty flood-control reservoir where outflow may be held to zero or a very small percentage of inflow.

9. The author's admonition to those tempted to use the published data for their own studies can be most appreciated by those familiar with the vagaries of nature in bestowing heavy rains at unexpected places and times. When combined with other peculiarities, floods result which can scarcely be predicted or provided for with statistical logic. For example, the designer of a spillway structure in the northern part of the United States must contend with fallen snow and ice; in the southeast the twin-peak flood gives emphasis to flood volume rather than peak flow; in Texas and other parts of the southwest, the 1935 floods show peak flows considered by some engineers to be "a phenomenon of a very special nature"; in California and the far west it is reported that peak flows have included a transient peak containing 50% water and 50% solids.[9] Engineers who must work with the more peculiar storms and watersheds will recognize the necessity for good judgment in determining their own design floods.

The writer's comments are intended to emphasize the author's own warnings concerning the use of data prepared for particular purposes. In almost every instance the provision of floodway capacity and flood storage is a matter involving dollars and sense. The engineer is guided, consciously or unconsciously, by the desire to make provision for expected and unexpected floods to the extent that increasing increments of flood protection do not require unreasonable outlays of money. If additional flood protection can be obtained at little or no cost, the engineer will be more than generous in providing such protection. If such additional expenditures are uneconomic from the financial and social viewpoints, the engineer will be tempted to restrain his imagination. The substance of the matter can be summed up as follows: Always provide sufficient protection for all reasonable contingencies and then add whatever additional protection that can be obtained without unreasonable added expense. Consciously or unconsciously, in matters not subject to mathematical precision, the engineer must be guided by dollars and sense.

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[9] "Transient Flood Peaks," by Henry B. Lynch, M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., November, 1939, p. 1623.

H. Alden Foster,[10] M. Am. Soc. C. E. (by letter).—For the last twenty years (1920–1940) the writer has followed with interest the various methods proposed for estimating stream flow in the design of reservoirs and dams. There has been a noticeable trend in the development of these methods, which may be worth considering. In 1914, the late Allen Hazen,[11] M. Am. Soc. C. E., made a revolutionary step in studies of hydrology by proposing the probability method for analysis of stream flow. This method received much attention from members of the profession and was extended in various directions.

Previous to the publication of Hazen's original paper, the late Weston E. Fuller,[12] M. Am. Soc. C. E., had introduced the idea of probability or frequency in the study of floods.

There have been numerous variations of Hazen's and Fuller's ideas, both in estimating future stream flow and determining maximum floods for design of structures. Fundamentally, however, all these methods were based on the assumption that, if a continuous record of stream flow or flood intensities were available covering a number of years, it could be used as a basis for estimating the probability of occurrence of a rate of flow or flood intensity of any given magnitude in the future.

The chief difficulty with the probability methods was that, in most cases, the available records were not sufficiently long to serve as a basis for reliable estimates of probability for periods exceeding the length of the available records to any great extent. Thus a probability curve based on a 10-yr record might be extrapolated to represent a 15-yr or 20-yr record; but it could not safely be extended to estimate the results of a 100-yr record. As few complete stream-flow records were available for periods of more than twenty to thirty years, the method did not seem to be of much use in some branches of hydraulic design, such as for flood control and particularly spillways of dams, where it was generally felt that the designs should be safe against floods whose probability of occurrence was less than once in one thousand years or more.

With a realization of this limitation of the probability method, efforts were made to develop a method for deriving a hydrograph of the "maximum flood" or the "maximum probable flood" which could be expected to occur on a river in a given location. Various ingenious methods were developed for this purpose. It is not the purpose of the writer to review these various methods, but only to indicate their trend. In recent years, the tendency has been to develop a synthetic hydrograph, based on a synthetic rainfall record. The rainfall record is supposed to represent that of the maximum storm which could occur over the watershed, or a storm of an intensity that would be expected at only rare intervals. In some of the methods, the principle of probability was used in deriving this maximum storm; in other methods (such as those described in the paper) the maximum storm is assumed to be greater than any on record but no attempt is made to estimate its probability of occurrence.

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[10] With Parsons, Klapp, Brinckerhoff & Douglas, New York, N. Y.

[11] *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

[12] *Loc. cit.*, p. 564.

Now it happens, in the United States at least, that rainfall records are available covering much longer periods of time than records of stream flow or of floods. It is practical, therefore, to apply the probability methods to rainfall records in many areas where a similar application to stream-flow studies would be quite unreliable. Such a combination of probability methods applied to precipitation studies with synthetic runoff records has been adopted by the U. S. Army Engineers, particularly in arid regions where stream-flow records are quite inadequate.

In the paper, the author has developed a very ingenious method for obtaining a "maximum probable storm," which he considers to be of sufficient rarity to deserve this description, but without attempting to estimate the probability of its occurrence in any year. By analysis of existing stream-flow records in the area under consideration, he develops a method for obtaining the 100% runoff hydrograph produced by this storm on a "standard watershed" of an assumed shape and size. To apply the method to any particular watershed he uses the following procedure:

(1) Prepare the storm hydrograph for the maximum probable flood on the standard watershed, corresponding to the actual drainage area of the watershed under consideration and the particular season of the year;

(2) Multiply the standard flood by a "location correction factor," which is supposed to correct for the variation of storm intensity at changing locations in the state;

(3) Multiply again by a "shape correction factor," which is supposed to adjust for the fact that the given watershed has a shape different from that assumed for the standard watershed; and

(4) Multiply again by a runoff coefficient, to make allowance for the fact that the actual runoff percentage will not be 100% of the rainfall, as assumed in preparing the standard hydrograph.

The final result will be a synthetic hydrograph of runoff in cubic feet per second per square mile; when this is multiplied by the actual drainage area in square miles, the final desired hydrograph will be obtained.

It will be observed that each of steps (1) to (4) involves the use of a quantity or factor which, in itself, is of uncertain accuracy. For example, if each step involved a positive error of 10%, the over-all error would be 47%. Even if the several steps should have margins of error considerably less than 10%, the final result might still be considerably in error.

The point that the writer wishes to emphasize is that any method which involves so many assumptions in its various steps cannot be expected to produce very reliable results; and since the final results cannot be considered precise by any means, the question may be asked whether equally good results might not be obtained by some less complicated method. The writer would suggest for such a simplified method that a probability analysis be made of the rainfall records to determine the ratio in magnitude and intensity of the "1,000-yr" storm (for example) to any given storm. Then the hydrograph of the flood produced by the given storm would be adjusted to obtain the runoff corresponding to the 1,000-yr storm.

The method used by the author for determining the variation of storm magnitude with geographical location is very interesting, and so far as the writer knows this method has not been described previously in print. The basic assumptions under which this method is developed appear to be sound, and very likely could be applied to advantage in other localities.

It is possible that the writer's impression of the complexity of the method would not be fully substantiated after practical application; familiarity often breeds respect. The paper is well worth study for its explanation of the various factors that influence the size of floods; and the writer is to be congratulated on the clear yet complete manner in which he has presented his subject.

EDGAR E. FOSTER,[13] **ASSOC. M. AM. SOC. C. E.** (by letter).—In this excellent paper the author has presented a method of devising synthetic design floods that is based essentially upon statistical analysis. The method is statistical in that the results are based on averages or extreme variations of a large number of observational data of the principal factors entering into the production of a flood. The resulting design floods, therefore, are not derived from one or, at most, a few great storms.

It appears to the writer that statistical analysis should constitute a sound basis on which to construct design storms, for the reason that hydrologic data are primarily statistics of the various elements of storms and floods. The method should be particularly adaptable where many design floods are required in a climatologically homogeneous region.

Storms and floods are complex phenomena, and in order to gain an adequate insight into them, they should be studied piecemeal. The primary elements of a storm are the rainfall, the duration, the area covered, and the location. Of these four elements, three are commonly expressed as numbers and therefore can be treated by statistical methods. Because of the great variation of the data, they can be conveniently expressed as averages and maxima. The fourth element, location, cannot so readily be expressed numerically, but it is the result of movements of the interacting air masses and depends largely upon the variation in atmospheric pressure and directions and velocity of the wind. Behind these four elements there are other more remote factors such as moisture content of the air and difference in temperature between the air masses, which play a part in the production of the storm and resulting flood. These factors may likewise be expressed as arithmetical data and are variable quantities, although the effects are not so readily traceable.

Still another element is necessary for the production of a flood even when heavy rainfall is received—that is, favorable ground conditions. This element is expressed as a numerical coefficient, being a ratio of the runoff and the rainfall, or as a rate of infiltration.

All these numerical data may be treated as statistics so that storms and floods can be analyzed into their constituent parts and averages obtained or envelope curves be drawn over the greatest observations to obtain probable maximum values. Since floods deal with maximum values, envelope curves are particularly valuable as shown in Figs. 2, 5, 7, 8, and 17. These averages and maxima can thereupon by synthesized into a design flood. This procedure is essentially the one the author has followed and is, in the opinion of the writer, a sound and logical process.

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[13] Head, Flood Control Section, U. S. Engr. Office, Omaha, Nebr.

The mechanics of the method probably will cause more disagreement among engineers. However, the same statement can be made of any other series of steps in developing design floods, for the reason that the art is as yet too young to have rigidly established procedures.

This situation applies particularly to the standard drainage areas as used by the author. This method of obtaining the distribution of runoff is empirical to a high degree and does not have a statistical basis comparable to that of the data of rainfall or other elements of storms. Like other formulas of similar nature, its use should be limited to the area and conditions under which it was derived. Furthermore, the derivation omits the effect of slopes and valley storage, elements which have an important bearing on the distribution of storm runoff.

Because there are a number of features in the paper that could be explained or harmonized by an understanding of the part played by the various air masses in the production of floods, it is regretted that the author did not include a more extended discussion of them.

It is desirable to trace the paths of the storms—that is, the movements of the areas of low barometric pressure, constituting the center of the extratropical cyclones. However, these areas of low pressure do not tell the entire story in themselves. The principal function of these "lows" is that they constitute a mixing pot for the various air masses in which the warm moist air is more or less rapidly lifted over the cold air so that the moisture content is precipitated with great intensity. Whether heavy precipitation occurs in the "low" depends upon the types of air being drawn or perhaps pushed into the center. For example, the "low" of the storm of March 16 to 18, 1936, entered the west coast of the United States on March 10 and produced little or no rainfall until March 16 at which time it was located over Louisiana. From that time on, it encountered the tropical air masses and heavy rainfall occurred. It should be noted in passing that the heavy rainfall which partly caused the heavy floods in New England occurred while the low was over the Middle Atlantic States; in this case the heavy rainfall cannot be accounted for except by movement of the tropical air masses over New England.

East of the Rocky Mountains, there are two types of air masses that are important in the production of heavy rainfall such as is required to cause a flood; these are the Polar Continental type (being named for the region of their origin) characterized by cold dry air, and the Tropical Maritime type that is the warm moist air of the Gulf of Mexico and adjacent tropical seas.

The latter type is the carrier of the moist air and the producer of the heavy rainfall necessary for floods east of the Rockies. The farther they travel from the source region, the more likely it is that the moisture content is depleted and the more likely that the supply of moist air will be interrupted or cut off by movement of the cold and dry polar air. In the United States this condition will not have a noticeable effect on the short storms such as thunder-storms during which moist air will be concentrated by convective action so that rainfall with a duration of a few hours may be nearly as intense in the northern states as in the southern. The great storms arising from air-mass action alone, however, require a continual supply of moist air to produce the precipitation observed, since the moisture content in a vertical column of air is not enough to produce the rainfall observed. As the tropical air masses move northward they leave their source of supply and at the same time approach the region of the cold air masses, which, being more vigorous in the northern latitudes, more readily cut off the supply of moist air. This results in a decreased total precipitation as the storms are located northward.

Figs. 1, 3, and 4 show the effect of the interrupted and diminished supply of moist air. Fig. 1 shows the steady decrease in rainfall along the axis of the Atlantic Coast of both summer and winter storms. The decrease appears (see Fig. 2) to be somewhat greater for the latter as may be expected because of the colder water surface that must be traversed from the source region of the tropical maritime air masses.

Likewise the storms moving northward from the Gulf of Mexico, shown in Fig. 5, have a diminishing total precipitation. This decrease is also shown by Figs. 3 and 4, along the southeastern extension of the axis. The peak (about mile 1300 on the Ohio Axis for summer storms) being located over the eastern border of Pennsylvania and New York, may be caused by the tropical air masses that come from the South Atlantic and enter land in the Middle Atlantic States. The dip in the axis in Fig. 4, between mile 800 and 900, remains unexplained unless the Gulf air loses its moisture before it moves eastward along the Ohio River Valley. Perhaps the storms are not susceptible of being sorted arbitrarily into summer and winter classes, or it may be that too few storms have been experienced to permit a certain classification. There seems to be no very good reason why winter storms should yield greater precipitation in western Pennsylvania than do the summer storms.

In any case the source of moisture is the same for both winter and summer storms, which fact accounts for the similarity in the two types of storms as noted by the author. There are some modifying influences, most important of which is the lower temperature of the gound that must be traversed in winter; this condition results in a smaller moisture content by the time the air masses reach the storm region.

C. S. Jarvis,[14] M. Am. Soc. C. E. (by letter).—The author has made a painstaking, logical approach toward acceptable evaluations of both flood volumes and their associated peak discharges, presumably for drainage areas ranging from 100 to 6,000 sq miles, but in practice readily adaptable to include much of the field outside those prescribed limits. For example, comparable values for 50 sq miles or for 10,000 sq miles may be readily derived by comparison and extrapolation, if the estimates for the prescribed range of 100 to 6,000 sq miles are well founded.

So many of the basic principles governing flood occurrence, magnitude, and probability have been utilized to such advantage in the author's presentation as to preclude challenge on details unless they are vital. One such exception must be noted, however, with respect to the statement (see heading "General") to the effect that these watershed factors (size and shape, location, imperviousness, slope, pattern, and nature of stream channels) remain the same for every flood on the watershed. Although the statement is accurate as to the first three factors—namely, size, shape, and location—the others are certainly subject to considerable alteration to conform with changes in land use, type and amount of vegetal cover, extent and degree of soil erosion, and the effects of corrective surface treatment designed (1) to lengthen the path traversed by surface runoff, (2) to reduce the gradients in the same proportion for the elementary channels, and (3) to alter the drainage pattern on the catchment area, before the runoff has concentrated into permanent channels of considerable capacity. As a result of changes in these three factors, a marked influence may be exerted on the imperviousness or, perhaps better, on the infiltration capacity of such drainage areas.

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[14] Hydr. Engr., SCS, Dept. of Agriculture, Washington, D. C.

Any one versed in the practice of irrigation can appreciate the significance of the foregoing comments by reference to the behavior of canal water as applied to newly reclaimed areas. If the water is allowed to follow the natural swales which may trend diagonally across a field, it will tend to erode the swale into a definite channel to permit flow on the steepest gradient, and to waste most of the water beyond the border of the field. However, the construction of guiding levees, terraces, straw or other dams, and regulating devices at suitable intervals will insure an equitable distribution into every furrow, flowing on gentle gradients for several hours, and retention of practically the entire volume of irrigation supply with little, if any, wastage of either soil or water.

Practical forestry, range management, and soil conservation measures today have as their objective an approach to the aforementioned condition for the conservation of available water, along with soil structure and fertility. To the extent that these conservation measures, including flood storage and detention, become effective and helpful in establishing permanent vegetal cover, either as forage or other perennial growth, a degree of protection and regulation is achieved, as has been demonstrated by numerous plots, fields, and small watersheds, and occasionally by larger units both in the United States and abroad. These surface treatments more or less offset the obviously negative effect of such disturbances of land surface as have attended man's occupation and infringements upon natural storage and overflow areas. Obviously, conservation measures have their limitations; but these have seldom been attained or closely approached.

It is reassuring to note the author's frank estimate of the intangibles connected with maximum flood estimates (see heading "General") and the cryptic expression, "The only certainty in regard to any future storm is that it will not exactly duplicate those of the past nor, for that matter, those developed herein." Such statements clarify the objective served by many ingenious devices and avenues of approach as used by the author.

The valley at 810 miles and the hump at 1,250 miles in Fig. 4 may appear somewhat irregular, but to the writer they seem quite reasonable as denoting, respectively, minimum storm depths due to remoteness from source of supply, and increased precipitation due to surmounting the Allegheny barrier ridges.

The map of storm paths in Table 1 seems to neglect the scope of the May–June, 1889, storm on the eastward slopes of the Appalachian Range; yet it is known that unprecedented floods of record, associated with the Johnstown, Pa., disaster, occurred on both the Potomac and the Susquehanna basins at that time, which continued as the record maxima until the March, 1936, floods established new records. Indeed, it may be impossible to depict a storm path satisfactorily by a single line, indicating merely the course followed by its center, when the disturbance is general over a width of several hundred miles, astride a prominent mountain range such as the Appalachian system. Should one not add some symbols to indicate width and scope of storm, as exemplified in Fig. 20? The added scale or index of width could be provided as shown at the northern terminus of storm path *A*, or at the southern ends of *B*, *C*, and *D* to indicate 200, 500, and 300-mile widths, respectively, with each added line representing 100 miles.

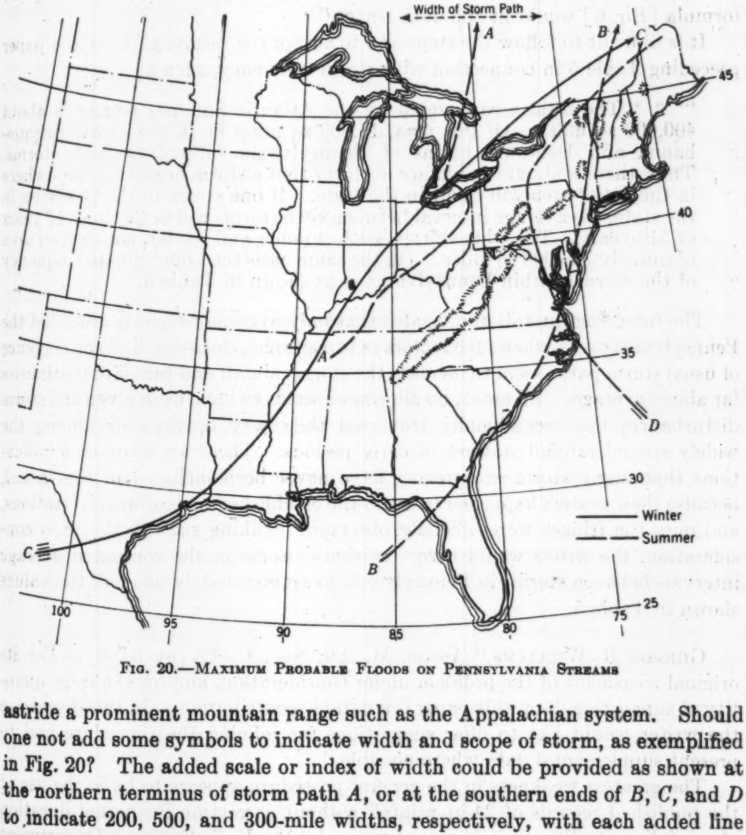

FIG. 20.—MAXIMUM PROBABLE FLOODS ON PENNSYLVANIA STREAMS

If the standard flood peaks vary as the 0.7 power of the drainage area (see heading "Development of the Standard Floods: The Standard Flood"), and the standard floods vary as the 0.3 power, then the mean variation is evidently in accordance with the 0.5 power. The writer's entire experience, confirmed by the graphs of this paper, is opposed to the 0.3 power relationship. In general, the slope of the envelope curve between 100 and 1,000 sq miles should be somewhat less than 0.5, rather than greater, indicating that the power is greater than 0.5, inasmuch as the relationship is inverse or reciprocal; that is, the slope measures the departure from unity, and, therefore, the greater the slope, the smaller is the fractional power. Furthermore, from the context following and the graph of Fig. 18, it is doubtful if any segment of the curve represents 0.3 power; it is more nearly 0.7 power. Quoting from the paper (see heading "Comparison with Flood Records"), furthermore: "Considering only the flood points, it appears that a steeper line, perhaps one represented by the formula [Eq. 6] would fit the data better."

It is difficult to follow the steps and to accept the results given in the paper preceding Table 5 in connection with storm frequency, such as

> "* * * The total area covered by the Atlantic Summer storms is about 400,000 sq miles. Of this area, 13,000 sq miles lie in the Lower Susquehanna and Delaware basins of Pennsylvania subject to such storms. Thus, there is about one chance in thirty that a storm occurring somewhere in the entire area will occur in the state. If one storm in thirty occurs in the state, the average interval between such storms will be 30 times 17 years or 510 years. The Ohio storms, both summer and winter, covered an area of roughly 780,000 sq miles. On the same basis the approximate frequency of the storms within Pennsylvania is as shown in Table 5."

The foregoing quotation indicates that only average weight is accorded the Pennsylvania areas, when such factors as topography, elevation, and convergence of usual storm paths seem to increase the storm rainfall and runoff potentialities far above average. Likewise, no allowance seems to have been given the storm disturbances that occasionally traversed their way between or among the widely spaced rainfall stations of early periods. There are abundant indications that many storm occurrences have never been included in the record, because their centers happened to avoid the established meteorological stations, and only the fringes were officially observed. Taking such factors into consideration, the writer would favor revision of some of the computed average intervals between storms in Pennsylvania to approximately one half the values shown in Table 5.

GORDON R. WILLIAMS,[15] ASSOC. M. AM. SOC. C. E. (by letter).—For its original treatment of the problem under consideration, and for the large quantity of supporting data, this paper is a notable contribution. In this discussion the writer would like to offer suggestions for refining the procedure and to present supplemental data where possible.

The greatest weakness in the general procedure appears to be in the use of the published records of 24-hr rainfall without correction for actual duration of rainfall. Studies made in recent years, by the U. S. Engineer Department in cooperation with the Hydrometeorological Research Section of the U. S. Weather Bureau, reveal the importance of determining actual durations of rainfall in estimates of maximum probable floods. A description of the method of conducting these studies is described in a paper[16] by G. A. Hathaway, M.

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[15] Associate Hydr. Engr., U. S. Engr. Office, Baltimore, Md.

[16] *Transactions*, Am. Geophysical Union, Part II, July, 1939, pp. 195–203.

Am. Soc. C. E. Briefly, the procedure is to determine from observers' original records the time of beginning and ending of rainfall, and from this information to construct mass curves of total rainfall. The mass curves for the non-recording gages are adjusted where necessary by comparison with mass curves from available recording gages and estimates of air-mass conditions in the region. The adjusted data furnish a basis for determining, closely, actual time-area-depth relations, which in many cases are much different from the relations indicated by published records of 24-hr rainfall. It is obvious that such investigations can be undertaken on an extensive scale only by organizations having large resources and trained personnel.

In view of the fact that the actual durations of most, if not all, of the rainfall in the storms listed as lasting 2, 3, 4, and 5 days were for much shorter periods, the use of the published durations to estimate maximum probable rainfall for short durations will lead to very large errors. As the results of the aforementioned studies have not been completed, information regarding actual durations of storms is very meager and is confined to point rainfalls only. Storm-duration studies of a long record at Boston, Mass., have been published by Charles W. Sherman,[17] M. Am. Soc. C. E.

As a generalization it may be stated that most of the rainfall from great storms in Northeastern United States has fallen within a period of about 60 hr or 2½ days. Published records may indicate that the period of intense rainfall lasted as long as 5 days. It is apparent that if the total rainfall is applied to unit graphs in 5 days instead of 2½ days there will be a great difference in the resulting flood discharges.

The U. S. Engineer Office in Baltimore has made an extensive study of seasonal rainfall at stations in the Susquehanna River Basin in Pennsylvania in connection with drainage problems for leveed areas. It may be safely assumed that the results of these studies of station rainfalls are comparable with the curves for 1 sq mile shown in Figs. 7 and 8. Fig. 21(*a*) shows the 100-yr depth-duration curve for the winter season (November to April, inclusive) for two recording stations in Pennsylvania compared with that of the author for 1 sq mile for standard winter storms, station 12, Ohio Axis. The curves for Scranton, Pa., and Harrisburg, Pa., are based on a frequency analysis for durations of from 5 min to 12 hr. Apparently, the curves presented by the author are too flat, and a suggested modification is shown as a broken line. Comparison of depth-duration-frequency curves with frequency curves for so-called 1-day and 2-day rainfalls indicates that the average duration of large winter rainfalls is about 16 hr for the 1-day storm and 24 hr for the 2-day storm.

Comparison of the curve for 1 sq mile for standard summer storms, station 8, Atlantic Axis, shown by the author, with an envelop depth-duration curve for Pennsylvania and northern Maryland is shown in Fig. 21(*b*). Again the curve given in the paper appears to be too flat.

The comparison of standard flood peaks with Fuller's formula is not significant except in a historical sense. The late Weston E. Fuller and his associate, the late Allen Hazen, Members, Am. Soc. C. E., were pioneers in the field of flood estimates, and they laid the foundations for much of the present development, especially in statistical analysis. However, more than 25 years have passed since the original Fuller paper was published (20), and in that time a mass of flood data, which would materially modify the original results,

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[17] *Civil Engineering*, March, 1939, p. 179.

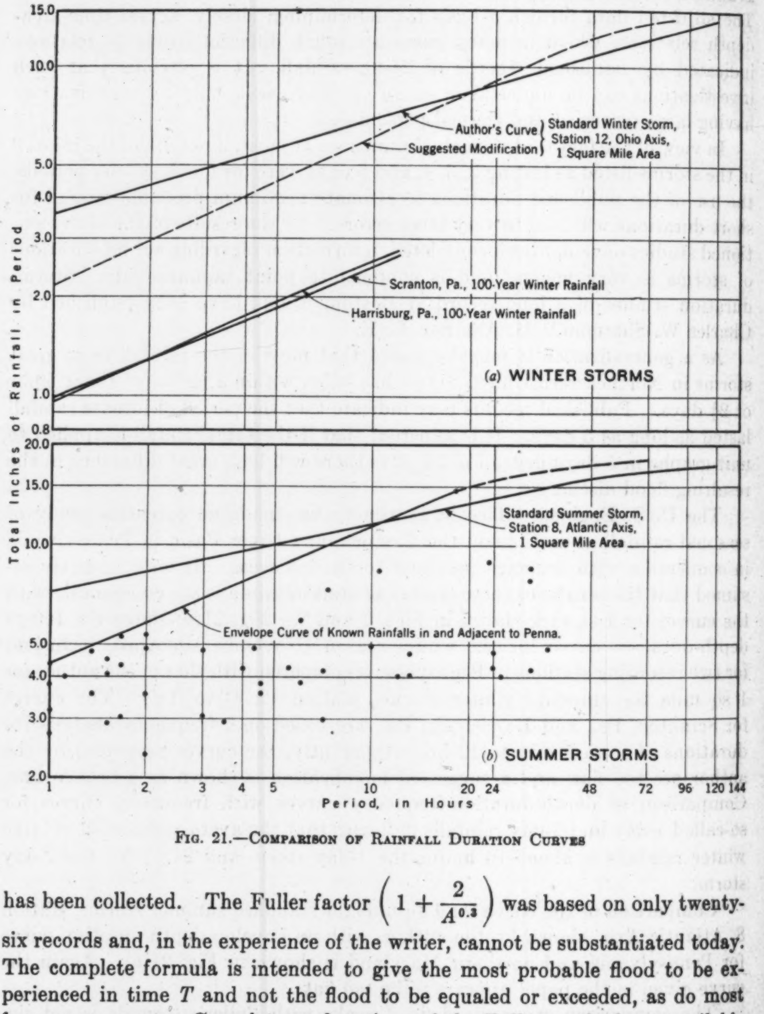

FIG. 21.—COMPARISON OF RAINFALL DURATION CURVES

has been collected. The Fuller factor $\left(1+\frac{2}{A^{0.3}}\right)$ was based on only twenty-six records and, in the experience of the writer, cannot be substantiated today. The complete formula is intended to give the most probable flood to be experienced in time T and not the flood to be equaled or exceeded, as do most frequency curves. Complete explanations of the Fuller method are available in a report of the U. S. Geological Survey.[18]

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[18] "Floods in the United States—Magnitude and Frequency," *Water Supply Paper No. 771*, U. S. Geological Survey, 1936, pp. 41–43, 46, 398–403, and 414.

As more data become available on infiltration rates, it will no doubt become advisable to substitute the method of subtracting infiltration from rainfall instead of using a runoff coefficient in determining the difference between rainfall and runoff in the maximum probable flood. Such a method is analogous to assuming a variable runoff coefficient throughout a flood and is more representative of actual flood conditions. Use of the infiltration method will tend to give higher peak discharges than the coefficient method.

It is the opinion of the writer that the relations between peak discharge and drainage area for the standard floods as shown in Fig. 14 give too flat a curve. That is, for areas less than about 200 sq miles the peak discharges are too small and for more than about 5,000 sq miles they are much too large. A curve represented by the equation $Q = \frac{5{,}000}{\sqrt{A}}$ closely represents an envelop curve of maximum known rates of discharge per square mile in Northeastern United States for areas of more than about 10 sq miles. Such a curve is defined by thousands of record-years[19] and, although higher discharges probably will be recorded many times, the slope of the line should be fairly well defined. A possible explanation of the flatness of the curves in relation to actual records, especially pronounced in Fig. 19, may be that the synthetic unit graphs are not applicable to the wide range of areas considered. The matter of the actual durations of rainfall, already discussed, will affect the composition of the standard storm and in turn affect the standard flood and perhaps the relation between the standard flood peak and drainage area. It might be expected that the curve for the maximum probable flood would be closer to the actual records for the large areas because the percentage differences between rare floods usually vary inversely as the sizes of areas.

The author expresses the belief that the maximum probable flood relation which he has derived represents floods that have an equal probability of occurrence throughout the range of areas considered, and therefore the derived curves shown in Figs. 18 and 19 would be flatter than an envelop curve for maximum known discharges. There is little but abstract reasoning to confirm or dispute such a conclusion. In this connection the writer has had occasion to plot curves representing the relation between discharges of equal frequency and size of drainage area for similar regions in Pennsylvania. These curves indicate that discharges of equal frequency vary between about the 0.5 and the 0.8 power of the area, with a tendency for the more infrequent floods to vary as the smaller power of the area. The sum total of hydrologic experience is still too limited to draw any reliable conclusions as to the possible relation between curves representing discharges of equal frequency and envelop curves of maximum recorded discharges.

The writer feels that there is opportunity for a vast amount of research in this very interesting problem of estimating maximum probable floods and that the author of this paper has laid the foundations for some valuable future developments.

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[19] "Maximum Discharges at Stream-Measurement Stations," by Gordon R. Williams and Lawrence C. Crawford, Assoc. Members, Am. Soc. C. E., *Water Supply Paper No. 847*, U. S. Geological Survey, 1941.

Emil P. Schuleen,[20] Assoc. M. Am. Soc. C. E. (by letter).—Ingenious methods are described in this paper for indicating the manner in which various factors controlling rainfall and flood discharge affect the magnitudes of the "maximum probable storm" and the "maximum probable flood." However, in view of the many assumptions that must be made in following the author's procedure to conclusion, his work should be more valuable in suggesting individual methods for assisting the engineer in making adjustments to such storms and floods as he may have under consideration for his particular problem than in providing direct values for the "maximum probable storm" and the "maximum probable flood."

The author's definition of the "maximum probable flood" is rather indefinite and raises the question: Just what is the "flood so large that the chance of its being exceeded is no greater than the hazards normal to all of man's activities"? His definition also suggests that he considers the term "maximum probable flood" as synonymous with the term "design flood." The writer believes that these two values should be considered separate and distinct, although the latter may be equal to the former in the case of large and important structures. The "maximum probable flood" should be the largest flood that is considered to be reasonably possible in the particular watershed, giving due weight to the basin characteristics and all other factors that would affect the magnitude of such a flood.

The "maximum probable flood," conceived in this manner, might then be used by the engineer as a basis for exercising his judgment regarding the design of the structure. For example, in the design of a channel improvement, the waterway opening under a bridge, or the spillway of a small and unimportant dam, the cost of providing discharge capacity sufficient to accommodate the "maximum probable flood" might well be prohibitive. On the other hand, in the design of a spillway for a large and important dam, where failure would result in loss of life and heavy property damage, the capacity should be sufficient to discharge, without danger to the structure, the greatest flood that might be considered reasonably possible in the particular watershed. The flood to be used as a basis of design, therefore, should be one resulting from a balance between the cost of providing a certain degree of discharge capacity and the danger in loss of life and property damage should failure of the structure occur. Such a criterion would naturally lead to the use of a flood equal to or closely approaching the "maximum probable" as a basis of design for a large and important project, but would permit the use of a flood of smaller magnitude in designing a less important project, where the hazard due to possible failure of the structure would be of minor consequence.

The author's study of variation in rainfall with respect to location along the two axes as reflected in Figs. 1 to 5, inclusive, should serve as a warning to engineers who might otherwise desire to translate storms over great distances without adjustment—as for example, from the Gulf or south Atlantic coasts to Pennsylvania. Since floods are the result of rainfall, the same warning applies equally well to the assumption that floods that have occurred in one region might occur in the same proportions in a basin of the same size and shape in another region widely removed from the first by distance or some physical feature, such as a high mountain range. As an additional caution in this respect, it might be mentioned that comparative studies of a large number of floods which have occurred somewhere in a region of relatively large extent will disclose several of unusual proportions for watersheds of their respective areas and shapes. Frequently, such records represent rugged basins with narrow valleys, steep hillsides, and steep stream gradients, which are particularly conducive to floods with high peak discharges. Records from such basins should not be considered representative of a flat or lightly rolling basin with wide valleys and sluggish streams, even though the areas and general shapes of the watersheds may be the same.

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[20] Senior Engr., U. S. Engr. Office, Pittsburgh, Pa.

The absence of intense summer storms along the Ohio Axis in the vicinity of station 8 may be the result of the deflection of storms originating in the east Gulf or along the south Atlantic Coast by the higher mountain peaks along the Tennessee-North Carolina border. However, it is noted in Fig. 5 that there is an apparent lack of data in the vicinity of station 8 covering summer storms of any magnitude. If this deficiency of data is characteristic also of the diagrams for the other values of area and duration, it may be that records from a larger number of stations in that vicinity representing a longer composite period would disclose values sufficiently great to make the enveloping lines for the Ohio Axis summer storms similar to the enveloping lines for the winter storms or the Atlantic Axis summer storms.

The rainfall values indicated by the duration curves of Figs. 7 and 8 appear to be low in comparison with records of storms that have occurred in the general vicinity of Pennsylvania. If the rainfall records used in deriving these curves represent once-a-day observations, it is quite possible that some of the storms may have been assumed to be of 24-hr or 48-hr duration, whereas the actual duration of rainfall might have been less than 24 hr with part of the precipitation being recorded as of one day and part as of the next. If this condition exists in the data, some of the storm values should be moved to the left in the diagrams and the duration curves in general thus would be raised. The possibility of such an error in assumption could be disclosed by analyses of the records from a number of stations for each storm by means of mass curves. This method of analysis, of course, assumes that the records from one or more recording rain gages in the vicinity are available to provide a "pattern," or that the time of beginning and ending of rainfall is available for one or more stations, or that the time of the once-a-day observation is not the same at each station for which records are used in the analysis.

There is no apparent reason why the curve for the Atlantic summer flood in Fig. 14 should differ considerably from the corresponding curves for winter and Ohio summer floods by continuing to vary as the 0.3 power in the range from 1,000 to 10,000 sq miles. In this respect, it is significant that the standard flood at this station as shown in Fig. 19 is considerably above the records of actual floods from drainage areas within that range.

It is noted that the curves of Figs. 18 and 19 tend to ignore actual floods which have occurred on small drainage areas. The author states that no correction was made for the runoff coefficient in plotting these actual floods. According to Fig. 17(*c*), the coefficients for the storms causing the summer floods probably were not greater than 80%, in which case the correct values for those floods in Figs. 18 and 19 would be greater than shown and therefore even farther above the standard flood lines. It is believed that these data indicate the standard flood lines to be too low for drainage areas up to about 1,000 sq miles.

This tendency to ignore or discount unusual values is also noticeable in Figs. 7 and 8, which in some cases apparently represent storms causing some of the intense floods on small drainage areas in Figs. 18 and 19. The author has indicated that many of these intense values are the result of very old records or of unofficial records of rainfall measured in buckets, etc. Although the writer agrees that such values may not be as reliable as later or official records, he believes that, in the absence of information to the contrary, they should be considered as reasonably accurate, after such checks as the engineer finds possible have been made.

The writer wishes to emphasize the author's conclusion to the effect that his results should not be "used blindly for design of flood-control projects or spillways." Curves and diagrams, such as those presented by the author, serve as an excellent guide for preliminary design or the check of a final design; but, in the case of a large and important structure, a careful study should be made of the storm possibilities over the particular basin, and the hydrograph of the "maximum probable flood" should be determined therefrom by means of a unit hydrograph for the specific basin, taking into account a runoff coefficient which might be considered reasonably severe, as well as an infiltration rate and a basic flow which might be considered reasonably possible of occurring coincidentally with the storm. The flood so determined might then be used as a guide for design.

WALDO E. SMITH,[21] M. AM. SOC. C. E. (by letter).—Inasmuch as the author has divided his comprehensive paper into two main parts, the writer will attempt to do likewise with his discussion.

With reference, first, to the part of the paper on flood-producing storms, Mr. Ruff has made an extensive survey of the location, duration, and extent of storms in the development of profiles along two axes, all of which work was done with the ultimate end of application to Pennsylvania. With regard to the Ohio Axis it would seem that, for general application, this should have been curved southward, conforming in the main to the Ohio-Mississippi channel with a Mississippi Axis continuing northward to which the Missouri and Iowa storms should be applied for profiles along that axis. This is in line with the author's statement that distances along the profile represent approximate distances from the normal sources of precipitable moisture. A sequence of isentropic charts which show the movement of tongues of moist, warm air inland from the Gulf of Mexico indicates that there is little justification for drawing the lower part of the Ohio Axis, as shown in Figs. 3 and 4. The effect of the selection of the position of the axes on the profile derived from the storms is shown in Fig. 22, in which the writer has replotted, with respect to the axes

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[21] Hydr. Engr., SCS, U. S. Dept. of Agriculture, Washington, D. C.

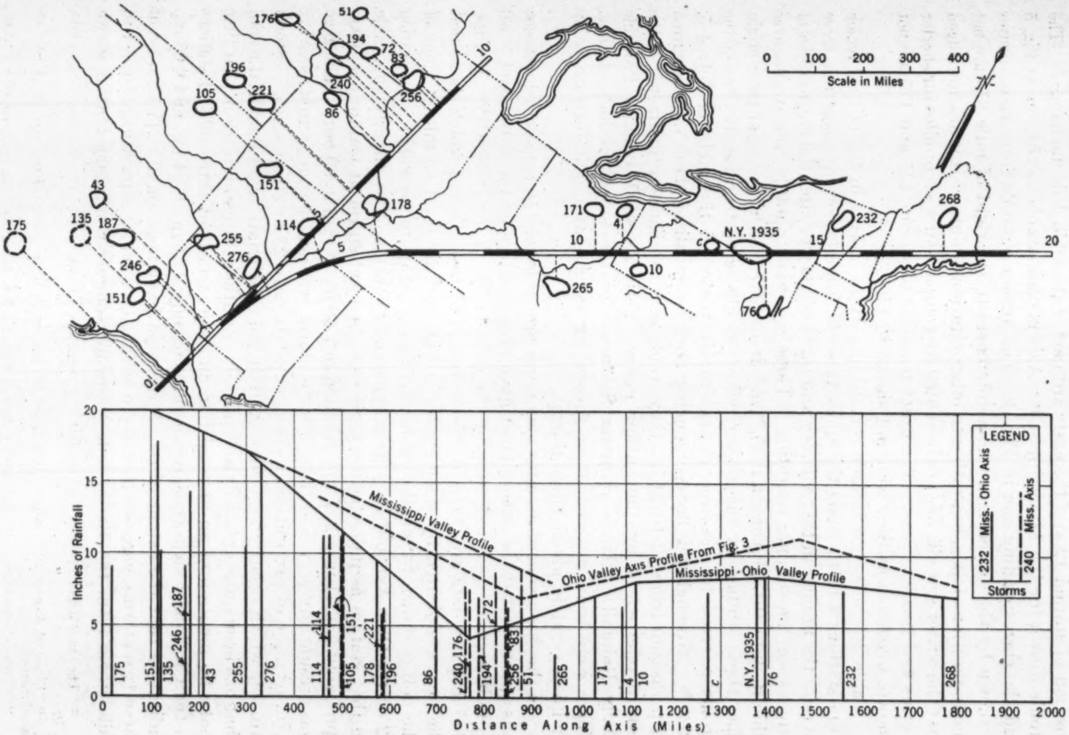

FIG. 22.—FIVE-DAY STORM RAINFALL ALONG THE MISSISSIPPI-OHIO VALLEY AND MISSISSIPPI VALLEY AXES IN SUMMER, ON 6,000-SQ MILE AREA

shown, the data in Table 3(*b*) for 5-day storms on 6,000 sq miles, holding the position of station 12 and the eastward part of the Ohio Axis unchanged. The profile drawn for the Mississippi-Ohio Axis is comparable to the one in Fig. 5 and the corresponding one in Fig. 3. The author's profile from those figures is shown by the dotted line. The differences to the right of mile 870 are not due to position of axis, but rather to interpretation of data. It is acknowledged that the writer's profile has not benefited by comparison with profiles for shorter storms and smaller areas. The exact positions of storms 135 and 175 are not known, inasmuch as they were not shown in Fig. 3.

Table 3(*b*) does not include certain important storms in the critical region from stations 8 to 10 along the Ohio Axis which might, on investigation, serve to raise the profiles in that vicinity and serve further to alter the entire series of profiles for the summer condition. These are, most notably, the storms over Ohio in July 1913,[22] August 1935,[23] and June 1937.[24] One other of later origin than those studied by the author, which would serve to raise sections of the profile referred to, occurred in eastern Kentucky on the night of July 4-5, 1939.[25] Although the flood-producing portions of all of these rains occurred in periods of from 12 to 24 hr, they would serve to cause adjustments of the nature referred to by the author under "Estimate of Maximum Probable Daily Rainfall: Atlantic Summer Storms," for the longer periods of time. There were some antecedent or succeeding rains that might increase 4-day and 5-day values above those shown.

In the discussion of data for maximum probable rainfall for short durations, it is apparent that the author must have prepared profiles for 1, 100, and 3,000 sq mile areas also, as indicated by the data in Fig. 7 for station 12. In Figs. 7(*b*) and 7(*c*) data are given for these areas, the inspection of which shows that the points indicated by small circles in Fig. 7(*a*) are for 1 sq mile. Many of the points shown by small dots in this figure are quite far afield from station 12 on the Ohio Axis but they were apparently considered as directly applicable to that station as indicated by their use. A local storm at Cambridge, Ohio (probably No. 41 in Fig. 7(*a*)), resulted in 7.09 in. of rain in 90 min. The gage was presumably very near the eye of the storm, whereas the entire area receiving ½ in. or more of precipitation was apparently only 25 or 30 sq miles.[26] Storm No. 44, which occurred on June 3, 1921, at Cincinnati, is an unofficial record that has been quite generally accepted.[27] Weather Bureau records of that time make no mention of an unusual rain and the records of three official stations in the Cincinnati vicinity show amounts of only 0.51 in., 0.34 in., and 0.05 in., indicating that the extent of the storm must have been small. The nature of these two storms suggests that there should be a wider separation of the 1-sq mile storm from the 100-sq mile storm for summer conditions than is shown by Fig. 7(*b*), especially for short periods of time. The uppermost curve for 1 sq mile should probably be substantially higher than is shown. This idea is further supported by the fact that, with spotty rainfall, it is rarely possible to measure the maximum rainfall with the wide scattering of rain gages that have been prevalent. This has been shown by C. W. Thornthwaite,[28] and is further supported by inspection of isohyetal maps[29] of intense summer storms of the Muskingum Climatic Research Project of the U. S. Soil Conservation Service. Very small centers of 5 in. or more in 4 to 6 hr are quite common, whereas areal averages of these amounts on 100 sq miles require longer periods of rainfall and are very few. Fig. 18 indicates that rainfall data from widely scattered stations alone give insufficient rainfall on small watersheds to account for the excessive flows sometimes observed, lending further support to Thornthwaite's contention. For areas in excess of 100 sq miles, the determined values of runoff seem to be reasonably consistent with the observed data, and the author is justified therefore in placing that value as the lower areal limit on his studies. With reference to large areas, and in line with Mr. Ruff's comment on this part of the work, the writer believes that probably an average of 2 in. of rainfall on 6,000 sq miles in 4 hr in summer is just about as rare an occurrence as in winter and that the left-hand ends of the lower summer curves of Fig. 7(*b*) should dip downward more sharply. In other words, there should probably be a much wider general divergence of the left-hand ends of the summer curves than is shown.

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[22] "The Ohio Water Problem," by the late C. E. Sherman, M. Am. Soc. C. E., Ohio State Univ., 1915, p. 10.

[23] "Flood of August 1935 in the Muskingum River Basin," *Water Supply Paper No. 869*, U. S. Geological Survey.

[24] "Climatological Data," U. S. Weather Bureau, Ohio Section, June, 1937, Bucyrus and other stations; also unpublished maps and other records, Climatological and Physiographic Div., SCS.

[25] Unpublished Report by the U. S. Engr. Office, Cincinnati District, Cincinnati, Ohio.

[26] "The Ohio Water Problem," by the late C. E. Sherman, Ohio State Univ., 1915, p. 16.

[27] "Cloudburst Phenomena," unpublished report by Ivan E. Houk, M. Am. Soc. C. E., presented before the Colorado Section, Am. Soc. C. E., June, 1925.

The author has used the term "inches of rainfall" frequently throughout his text and on figures without indicating whether this is "inches or more" as is used in various studies, or an average value. It has been interpreted by the writer as being the latter.

Reference is made to the term "duration in days" which the writer has presumed to mean the number of consecutive days on which the rainfall indicated was recorded, rather than actual duration, which for a recorded 2-day rain may have been a period varying from a very short time to 48 hr with an approximate average of about 24 hr; and for a 3-day record of rain may have been in a period ranging from a little more than 24 hr to 72 hr with 48 hr as the approximate average, as shown by C. W. Sherman[30] and G. A. Hathaway,[31] Members, Am. Soc. C. E.

With reference to the second part of the paper on standard floods, the author has indeed developed an interesting and comprehensive method that would seem to give consistent results for a given region. Area-correction and shape-correction factors are especially well treated. He has been wise to counsel against the use of this method for design purposes, without taking proper account of the numerous other factors involved that do not appear in their proper weight in such an analysis.

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[28] "The Reliability of Rainfall Intensity Frequency Determinations," by C. Warren Thornthwaite, *Transactions*, Am. Geophysical Union, 1937, pp. 476–484.

[29] An example is found in "Reliability of Station-Year Rainfall-Frequency Determinations," by Katharine Clarke-Hafstad, *Proceedings*, Am. Soc. C. E., November, 1940, p. 1608.

[30] "Actual Duration of 'One-Day' and 'Two-Day' Rain Storms," by Charles W. Sherman, *Civil Engineering*, March, 1939, p. 179.

[31] "Estimating Maximum Flood Flow as a Basis for the Design of Protective Works," by Gail A. Hathaway, *Transactions*, Am. Geophysical Union, 1939, pp. 195–203.

The writer has found that lag time from the center of mass of the rainfall producing runoff to the center of mass of the runoff, as suggested by W. W. Horner, M. Am. Soc. C. E., and F. L. Flynt,[32] Assoc. M. Am. Soc. C. E., is generally a more useful term than the crest lag used by the author; and it is less subject to variation on a given watershed for various amounts of rainfall, within a given time unit, than the crest lag.

The writer frequently has found that, for many purposes, the data given in Fig. 12 showing individual values for the distribution graph may be more conveniently plotted as a summation graph as shown in Fig. 23. The particular advantage of this form is that one can always select values the first time that total 100%, although the presentation as given in Fig. 12 has the advantage that the peak period is always obvious.

Under the heading "Development of the Standard Floods: The Standard Watershed," Mr. Ruff states: "The graphs would be equally applicable to any watershed having the same lag." This sentence should have been qualified in accordance with practice elsewhere, with reference to its application to the Appalachian highlands.

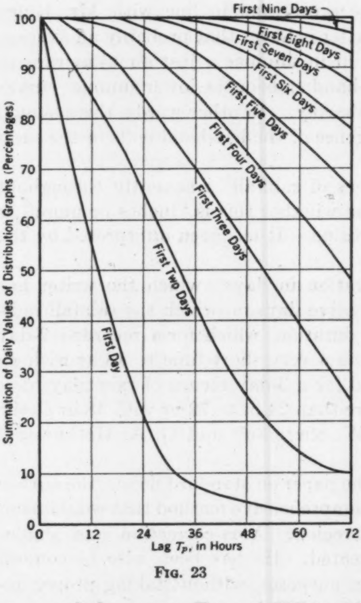

FIG. 23

Again, under the heading "Development of the Standard Floods: The Standard Flood," the statement is made that:

> "The standard floods vary closely as the 0.3 power of the watershed area for areas between 100 and 1,000 sq miles, and the total peak flow varies as the 0.7 power of the area. From 1,000 to 10,000 sq miles, the Atlantic summer flood continues to vary at this rate, but the winter floods vary more nearly as the 0.4 power, and the Ohio summer flood as the 0.45 power of the watershed area."

In other words, the crests of standard floods (in cubic feet per second per square mile) vary inversely as the 0.3 power of the watershed area for areas between 100 and 1,000 sq miles, or the maximum probable flood in terms of cubic feet per second varies as the 0.7 power of the area. From 1,000 to 10,000 sq miles the Atlantic summer flood continues to vary at this rate, but for the winter floods, the peak per cubic foot per second per square mile values vary more nearly as the inverse 0.4 power, and the Ohio summer floods as the inverse 0.45 power of watershed areas.

It seems undesirable that after all the care that the author has taken to arrive at rational values, it is then necessary to seek an arbitrary runoff coefficient to apply to the results in order to arrive at the desired answer. Still, the writer has no alternative to offer for winter storms. However, the factor 120% impresses the writer as being small. The runoff from winter storms on bare ground in the northern Appalachian highland region frequently approaches 100% for large storms. The 20% then, roughly, is the allowance for melting snow and ice. On an area on which 5 in. of rain in 24 hr produces the maximum winter flood this allowance is only 1 in. If the March, 1936, flood on the Pennsylvania headwaters of the Ohio River had not been preceded by a mild period a few days earlier which greatly reduced the snow cover, the resulting flood stages would have been much higher. In Fig. 5 the rainfall causing the March, 1936, flood is shown for a 6,000 sq mile area as having a value of about 7 in., whereas the profile indicates about 8½ in. Thus, for this particular area and storm, and for a 5-day duration, there would be an allowance of 1½ in. plus 20% of 8½ in. for the melting snow cover, which seems adequate; but for smaller areas or other durations the question still remains.

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[32] "Relation Between Rainfall and Run-Off from Small Urban Areas," by W. W. Horner and F. L. Flynt, *Transactions*, Am. Soc. C. E., Vol. 101 (1936), p. 140.

With regard to summer floods, the 90% factor seems too high. One might find a new approach to this value through an application of infiltration values, much data on which are now becoming available in Pennsylvania. Before leaving the subject of runoff coefficient the writer believes that, fortunately, the author has twice refuted his opening general statement that imperviousness is the same for every flood on a watershed. He first refutes this idea in the second sentence following its statement when he declares that "The condition of the ground at the time the storm occurs * * * is also a factor," and again by inference in the methods of analysis used for the modification of runoff coefficient because of season of the year. It would be well for hydrologists and hydraulic engineers to cease considering degree of imperviousness as a watershed constant, inasmuch as overwhelming evidence shows that it varies not only from season to season, and storm to storm, but within a storm period.

Charles F. Ruff,[33] M. Am. Soc. C. E. (by letter).—Mr. Benson, and several others of those who discussed this paper, have asked for a more specific definition of the "maximum probable flood." In referring to the size of floods in this sense the conception of frequency is involved. A given flow would not be a "large" flood if it occurred every year; although on another stream the same flow might be considered a very large flood because it had occurred only once in a long record. Mr. Alden Foster states that spillways should be safe against floods whose probability was less than once in a thousand years, and the writer would accept the 1,000-yr flood as the maximum probable flood if several thousand years of record were available from which it could be estimated. The writer concurs in Mr. Schuleen's excellent statement as to the use of this flood; making the design flood larger or smaller as the cost and importance of the structure warrant.

Some idea of the size of the maximum probable storm (and therefore the flood derived from it) can be gained by comparison with the storms used in designing various projects.

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[33] Engr., Federal Power Comm., Washington, D. C.

The "Official Plan" storm of the Miami Conservancy District, the storm against which the works were designed to furnish protection, was taken as 10 in. of rainfall in three days. The area above Dayton, Ohio, the principal damage center, is 2,500 sq miles. The value of the writer's maximum probable storm for this location, duration, and area is 10.5 in. For design of the spillways of the reservoirs the rainfall was taken as 14 in. in three days. The drainage areas above the reservoir ranged from 255 to 1,135 sq miles, and the writer's storm for these areas ranges from 13.5 to 12.4 in. in three days.

The Muskingum "Official Plan" storm rainfall for various areas and durations was about two thirds of the writer's maximum probable storm for this location. The spillway design storm, for 100 sq miles and three days' duration, was 15.2 in. as compared with 12.9 for the maximum probable storm. For areas of 500, 1,000, and 2,000 sq miles the spillway design storm is slightly smaller than the maximum probable storm as derived by the writer.

The summer storm used by the Pittsburgh office of the Army Engineers in their 1928 report was about 1 in. less than the writer's for various areas and durations; the winter storm was about three fourths of the writer's on 500 sq miles and about equal on 6,000 sq miles. The Baltimore office of the U. S. Engineer Department used, for spillway design on the York, Pa., project, a summer storm about 7% higher than the writer's for an area of 100 sq miles and durations of 36 to 96 hr. The winter storm used was 40% higher for the shorter duration and about 10% higher for the 96-hr duration.

Maximum rainfalls predicted by the U. S. Weather Bureau for various watersheds in the Susquehanna basin in New York matched the writer's maximum probable storm within 10% for winter, some being lower and some higher. The summer storms were generally near the Ohio summer storms, although some approached the Atlantic summer storms, which are slightly larger.

These comparisons indicate that the maximum probable storm is somewhere near the size of the storms that have been used or proposed for spillway designs; and generally somewhat larger than the storm whose flood is to be completely controlled.

Mr. Benson questions the propriety of rotating the storm. The writer agrees that the direction of the longer axis of the watershed would have some effect on the amount of average rainfall but left this, along with the effect of local topography, among the factors that were not accounted for. In a study of Pennsylvania rainfall the writer endeavored to investigate the general shape and position of storms by the following method:

A circle and two ellipses, one with a ratio of major-axis to minor-axis of 2 to 1 and another with that ratio 4 to 1, were drawn on tracing cloth at such a scale that the area enclosed by each represented 3,000 sq miles on a map of the state. Values of rainfall at all stations for each storm of record having more than 4 in. of rain in three days were plotted on the base maps of the state. The figures described were then placed over the map and adjusted to a position that gave the highest average for the rainfall stations included inside one or the other of the figures. Nearly half the storms had an area of highest rainfall best covered by the 2-to-1 ellipse. The other half was about equally divided between the circle and the 4-to-1 ellipse. The major axes of both types of ellipse lay in a general northeasterly direction, ranging from north to east. None of them lay in the northwest quadrant. Thus the maximum probable flood from an elongated watershed lying northwest and southeast might be expected to have a somewhat smaller value than that for one lying northeast and southwest.

The writer agrees with Mr. Alden Foster's statement regarding the limitations of probability methods due to the shortness of the records. The suggestion that the use of rainfall data would permit determining the 1,000-yr rainfall seems open to the same objection. Rainfall records are generally no longer than 50 years (since about 1889, in Pennsylvania). Historical flood records at many points go back 100 or 150 years, at least for the larger floods. A recent paper by Mrs. Katharine Clarke-Hafstad has discussed[34] the probable error in rainfall probability based on station-year analysis, and showed the error for the 100-yr rain to be of the order of ±30%. It would doubtless be considerably greater for the 1,000-yr rainfall. The determination of the 1,000-yr average rainfall on the watershed, as suggested by Mr. Foster, would correspond to steps 1 and 2 of the writer's method. To reduce this rainfall to flood flow a number of methods can be used: Analogy with a known storm and flood, as suggested by Mr. Foster; a unit hydrograph and runoff coefficient or infiltration rate (which would perhaps be most accurate); or the writer's standard flood and shape correction. However, in all these methods assumptions are either made or implied as to the effect of the shape of the watershed, the amount of rainfall that runs off, and the possibility that such a rainfall can occur.

Mr. Edgar Foster's discussion gives a very good explanation of the action of the moisture-bearing air and its effect on the maximum rainfall at different points. He questions the dip in the summer rainfall envelop in Fig. 4 between mile 800 and 900. The writer feels that this dip may be too deep, due to incompleteness of the data, but agrees with Mr. Jarvis that such a dip is not unreasonable. One reason for the lesser summer rains in this area which suggests itself is that the moisture-bearing air (not to be confused with the low pressure areas) travels in from the Atlantic to this area. In summer such air is unstable, and the lifting due to crossing the mountains precipitates most of its moisture. In winter it is relatively more stable and the precipitation occurs only when it meets colder air masses farther inland.

Mr. Jarvis questions whether the watershed factors of imperviousness, slope, pattern, and nature of stream channels are constant and cites the effect of forest management and soil conservation measures. On areas of the size dealt with in this paper a tremendous program of such measures would be necessary to affect, materially, the character of the maximum probable flood (smaller floods might be more easily modified) and such effects along with possible dam failures and ice jams were left for separate consideration. As between a steep rocky watershed and a flat sandy one there will be a permanent difference in imperviousness as well as the variations due to the season and the culture.

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[34] *Proceedings*, Am. Soc. C. E., November, 1940, p. 1603.

Some indication of the width of the storm path, as suggested by Mr. Jarvis, would be desirable, but would require considerable research of the weather maps. Mr. Jarvis states that "If the standard flood peaks vary as the 0.7 power of the drainage area * * * and the standard floods vary as the 0.3 power, then the mean variation is evidently in accordance with the 0.5 power." This statement seems to be based on a misunderstanding, for which the writer is partly to blame. The standard flood q, in cubic feet per second per square mile, varies nearly inversely with the drainage area, thus

$$q = \frac{K}{A^{0.3}} \quad \text{.......................} (7)$$

Multiplying Eq. 7 by the drainage area A, the total peak flow Q, in cubic feet per second, is

$$Q = q\,A = K\,A^{0.7} \quad \text{.......................} (8)$$

Thus it is improper to average the exponents 0.3 and 0.7. The estimates of the frequency of occurrence of the maximum probable storm in Pennsylvania was made only to indicate the rarity of such storms, and they could be cut in half, as Mr. Jarvis suggests. However, even when the storm occurs within the state it must still center exactly over the watershed under consideration in order to produce the maximum probable flood on that stream, so that the intervals between such floods on a given stream would be many times longer than the intervals between storms occurring somewhere in the state.

Mr. Williams emphasizes a weakness of deriving the maximum probable flood by describing the difference between rainfall records for calendar days and the actual rainfall hour by hour. The writer appreciated this difficulty, but could find no data for hourly rainfall comparable to the "Miami Report" for daily rainfall. For the largest area of standard watershed computed (6,000 sq miles) the critical period of rainfall is about 6 hr (Table 6) and this rain accounts for 76 cu ft per sec out of the 122.5 cu ft per sec peak flow (Table 7, Items 10 and 11, Curve *E*). The rainfall used to develop the standard flood, in periods of 6 hr or less, was taken from the rainfall-duration curves of Figs. 7 and 8, and not directly from the calendar-day rainfall records. Mr. Williams states that these curves appear too flat, and he submits a steeper curve (Fig. 21) with less rainfall for the critical periods of 1 to 6 hr. Thus the writer feels that he has made ample allowance for the difference between calendar-day and hourly rainfall, as pointed out by Mr. Williams, and is perhaps on the high side in this respect.

Mr. Williams suggests that the curves of standard floods (Figs. 18 and 19) are too flat—that is, too nearly horizontal. If the writer revised his rainfall-duration curves, as suggested in Fig. 21, using the lesser rainfall for the short durations which are most critical for the small areas, the standard flood for these areas would be relatively smaller, and the standard flood lines in Figs. 18 and 19 even flatter. On the other hand, to make these flood curves steeper would require higher rainfall for small areas and short durations than those the writer used, which Mr. Williams already finds too high.

It should be emphasized that the standard flood curves are derived entirely from rainfall; in fact they were developed and plotted as in Fig. 14 before it occurred to the writer to compare them with actual floods. The standard watershed may be regarded as a special type of rain gage which reduces the three dimensions of the storm—area, depth, and duration—to two dimensions of area and flow.

Mr. Schuleen also mentions the difference between actual duration of rainfall and the apparent duration of rain from once-a-day observations. Such observations were not used without the aforementioned allowance being made.

The ignoring or discounting of a few unusual values was deliberate. No criticism of the accuracy of the old records is implied, since for this purpose the nearest half inch of rainfall would be good enough. However, in deriving floods that could still be considered probable (not merely possible) it seemed that the rare and extreme rainfalls might be ignored. Mr. Schuleen gives a description of how the "maximum probable flood" should be used which the writer considers to be excellent.

Mr. Smith's extension of a Mississippi Axis seems reasonable for investigation of that part of the United States. With only Pennsylvania in mind the writer did not investigate such a possibility. The storm paths used to indicate the location of the axes, some of which are shown in Table 1, were only for those storms that caused floods in Pennsylvania. Had the storms been picked with reference to floods in the Mississippi Valley a somewhat different picture might be presented. The differences in the two profiles shown in Fig. 22 are (as Mr. Smith states) a matter of interpretation. The writer made profiles for each duration (one to five days) and each area, and the height of the two-day, three-day, and four-day profiles was considered in drawing the five-day profile. Storm No. 76 was a two-day storm (see Table 3). The profile, suggested by Mr. Smith for the five-day storm, just touches the top of it. The writer's profile is higher to make allowance for possible rainfall in the other three days. Rainfall for four days of the NY 1935 storm was nearly an inch more than for the two maximum days.

In showing rainfall for "1 sq mile" the writer followed the terminology of the "Miami Report," which designated the reading at a single station as 1 sq mile (in comparison with 500, 1,000, and 6,000 sq miles). Actually, as Mr. Smith shows, the reading at one gage may vary considerably from the average over 1 sq mile. Perhaps the designation "0 sq mile" would be preferable.

The term "inches of rainfall," as used, means the average value over the area in question as Mr. Smith assumes.

The runoff coefficient of 120% for winter storms is intended to include a "probable" amount of snow, and not the maximum possible. As Mr. Smith states, it would amount to only 1 in. for a 5-in. rain. However, for this purpose the important factor is not the total snow cover, but the amount that will melt and run off during two or three critical periods which, for the largest areas considered, may amount to only 18 or 19 hr. The release of 1 in. of runoff (which would be equivalent to 8 or 10 in. of unmelted snow) in this time seems to be ample.

Several discussers have mentioned the superiority of the use of infiltration rates and computed water losses as a means of determining runoff instead of using a runoff coefficient. The infiltration method is particularly valuable for computing the runoff from a storm where the rainfall is variable and intermittent. However, the writer would not attempt to predict any particular pattern for a hypothetical future storm, and considers that the use of a runoff coefficient which varies with the size of rainfall and season of the year is all the refinement warranted by the approximate nature of the other assumptions that must be used in such a study.

TABLE 11.—COMPARISON OF SEPTEMBER, 1938, RAINFALL WITH MAXIMUM PROBABLE STORM (RAINFALL IN INCHES)

Area, in sq miles	Duration, Two Days		Duration, Four Days	
	Observed	Fig. 1	Observed	Fig. 1
500	11.0	11.0	15.5	12.0
1,000	10.5	10.5	14.8	11.5
6,000	9.0	9.6	12.3	10.3

A further idea of the size of the maximum probable flood is gained from a comparison with the New England hurricane floods of 1938. Data on this storm were not available to the writer at the time the paper was submitted, early in 1939, so it constitutes somewhat of a test of the method. The data used are from *Water Supply Paper No. 867*, published in 1940.

The actual daily rainfall compared with the writer's maximum rainfall as read from Fig. 1 for New England is given in Table 11. None of the hourly readings approached the lines in Fig. 8. It will be observed that the four-day rain exceeded the maximum rainfall used by the writer. This storm is thus one of those larger storms mentioned by the writer (see paragraph preceding "Maximum Probable Rainfall for Short Durations") which may be expected to occur, but which is supposed not to center over a particular watershed in such a way as to exceed the maximum probable flood.

TABLE 12.—COMPARISON OF MAXIMUM PROBABLE FLOOD WITH NEW ENGLAND FLOODS OF SEPTEMBER, 1938

(Runoff Coefficient from Fig. 17 for 60° F Temperature and 9-In. Rain)

No.	Gaging station	Area, in sq miles	Standard flood[a]	Correction Factors, %			Maximum probable flood[a]	Observed peak[a]	Ratio, Col. 9 / Col. 8
				Shape	Location	Runoff			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
163	Cold River above Black Brook....	23.4	670	133	83	87	640	624	0.97
166	Chickley River at West Hawley...	25.1	660	119	83	87	567	709	1.25
164	Cold River below Black Brook....	30.1	621	122	83	87	548	561	1.03
143	Rock River at Williamsville......	42.6	565	125	82	87	503	638	1.27
159	Deerfield River at Charlemont....	159[b]	385	117	83	87	324	352	1.08
160	Deerfield River near Charlemont..	178[b]	375	117	83	87	316	316	1.00

[a] In cubic feet per second per square mile. [b] Exclusive of area above Harriman Reservoir.

The U. S. Geological Survey reports peak flow at 532 points on New England rivers in the area covered by the hurricane. Of these, four exceeded, one equaled, and one was only slightly below, the maximum probable flood, as computed by the writer's method. The six floods are shown in Table 12. All are less than 500 sq miles in area, and all but two less than 100 sq miles. Although the method is not proposed for areas of less than 100 sq miles, had it been used to design 532 dams at these gaging points only four of them would have failed. Considering the unusual nature of the hurricane, and the tremendous damage done to all kinds of structures, this record does not seem to exceed the hazards normal to all the works of man.

In closing, the writer wishes to express his appreciation of the many good points revealed by the various discussions which he has not had opportunity to mention.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 2127

TOTAL WAR AND THE ENGINEER

ADDRESS AT THE ANNUAL CONVENTION, SAN DIEGO, CALIFORNIA, JULY 23, 1941

By Frederick H. Fowler,[1] President, Am. Soc. C. E.

That engineers have been present on these coasts of California since the earliest days is definitely recorded in Father Crespi's journals of the Portola Expedition, wherein are given the latitudes of the successive camp sites on the march northward as observed by the good Father, and by "an Engineer."

It was on this expedition also, if my memory serves me, that local phenomena later of import to the engineer were first observed, and Christian units in the precise measurement of time first introduced into the heathen land. "This night," writes the Reverend Father, "we encamped beside the Rio Santa Ana, and experienced a most terrifying earthquake. The first shock lasted about half as long as an Ave Maria * * *." Sad to say, the observations of "the Engineer" are lost to history.

Further evidence that civil engineers have been present from Spanish days is found in the remains of the Mission's irrigation works, which show that engineering has been practiced from the earliest times, even if by individuals without collegiate degree, and lacking Certificate of Registration from State or Crown.

Even at the time of my first visit, the San Diego of fifty years ago, while retaining much of the sunny calm of Spanish colonial days, had been modernized by the advent of a single-track railroad—the work of an Engineer—and the back country had been assured a more stable water supply, made available by the Sweetwater Dam, then recently completed. This dam, with its "height 94 feet"—a wonder of the modern world—was designed and constructed by the late James Dix Schuyler,[2] later a Vice-President of this Society, and was worthy of inspection by all and sundry, in the same manner that we visit the 726-ft Hoover Dam of today.

In brief, the full energies of the civil engineer, from Colonial days to the time of my first visit to San Diego in the middle eighteen-nineties, and for nearly a quarter of a century thereafter, were concentrated on the development and betterment of an arid land. Only during the first World War, with the completion of Camp Kearney, on the mesa nearby, did the program change from building for constructive progress to building for destruction.

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[1] Cons. Civ. Engr., San Francisco, Calif.

[2] For memoir, see *Transactions*, Am. Soc. C. E., Vol. LXXVI, December, 1913, p. 2243.

But even then we visualized a definite end to strife, since the cry in 1917 was that we were waging a "war to end war." My faith in this slogan was somewhat shaken when, on a week-end leave from training camp, I chanced to open a translation of Cicero's oration on Scipio Africanus Minor, and read therein something to the effect that: "Scipio, by destroying our two great adversaries, Carthage and Numantia, ended war for all time."

Today, a glance around this erstwhile peaceful city of San Diego will show that neither by the efforts of Scipio Africanus in the second century B.C., nor by our own of 1917–1918, has war been ended. In fact, a survey of our immediate surroundings, the country as a whole, and the entire world, will show that we have progressed—or from a humanitarian standpoint, have retrogressed—to that most destructive of all things, the most modern of all martial inventions, "Total War."

By "Total War" we do not merely mean that we concentrate on the enemy our heaviest battalions afield and our heaviest broadsides afloat. "Total War," it has been said, is the organization and exploitation of every resource and device, the employment of the energies of every individual within the warring state; it is by the demands of total warfare that today every mine, forest, field, and factory, and every man and woman in the country are being brought by an inevitable process into active service.

Progress in this new type of war is measured by units of destruction rather than units of production. "Man-hours" of productive labor give way to "Division-days" of annihilation—the destructive work of one Army Division for one day.

Poland, we are told, was conquered within thirty days at a modest figure of 1,050 "Division-days." France, the Netherlands, and Belgium fell after forty-five days at an expenditure of 9,000 "Division-days," and altogether the Nazis invested only 11,550 "Division-days" in conquering the major part of Europe.[3] If all their Divisions had fought every day their output would have been 144,000 "Division-days"—an index of potential destruction still in store, through concentration.

It is clear that such destruction could be accomplished in so brief a time only in a mechanized age, and a mechanized age must needs be an engineering age. It therefore behooves us to make some appraisal of what we as individuals, and as members of the American Society of Civil Engineers as a whole, have accomplished in preparation for a continued "all-out" effort for our part in "Total War."

CONSTRUCTION ADVISORY COMMITTEE OF THE ARMY AND NAVY MUNITIONS BOARD

The Society's own activities in the present emergency, first brought to the attention of the membership in President Hogan's address at Denver, Colo., last July, have been in brief as follows: In the early part of 1940, realizing that the country's preparedness program and the engineers' part therein were at least a year behind, we, together with the other Founder Societies, the Architects and the Contractors, initiated cooperation with the Army and Navy Munitions Board. On May 6, 1940, this hitherto informal organization was formally appointed by the War and Navy Departments as the "Construction Advisory Committee of the Army and Navy Munitions Board." It consisted of the heads of the various organizations represented, and five out of its six members were members also of the Society. The Advisory Committee, so constituted, has since confined itself, with one exception, to questions of policy, and concentrated largely on establishment of proper conditions of contract and employment.

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[3] *Fortune*, July, 1941, p. 76.

These particular activities were initially directed toward the passage of proper legislation; the elimination of agency contracts to manufacturers to design, construct, and operate; establishing proper forms of construction contracts approved for management; segregation of engineer-architect contracts from construction contracts; the elimination of competition in fees; establishing regulations and fees for all subcontracts; establishing the principle of utilizing existing engineering and architectural firms instead of expanding existing government organization; urging the widest geographical distribution of technical and construction contracts to avoid overloading and favoritism; preventing undue migration of technical and construction personnel; insuring adequate compensation for engineers and architects under various classifications; and insuring adequate fees for consulting services.

The work of the Advisory Committee was largely completed by the end of last year. Summarizing the results very broadly, it was found possible to extend to other branches of the government beneficial legislation already passed for the Navy; agency contracts could not be entirely eliminated; proper forms of contract were developed in the Construction Division, Office of the Quartermaster General, and in the U. S. Engineer Department, of the Army, and had already been established in the Navy; separate engineering and construction contracts were established at the outset in the Army, and later by change of policy in the Navy; little trouble was encountered in competition for fees; regulation of subcontracting was secured in the Army and, although initially unsatisfactory, was later remedied in the Navy; building up of government organizations has been limited very generally, but some forces of considerable size already existing have not been reduced; a far more general geographical distribution of work has been secured; it is probable that no abnormal migration of technical personnel has taken place, although men are snapped up as soon as released and it is difficult to recruit a force for any new project; adequate compensation for engineers and architects is in effect; the adequacy of fees depends upon the department, and upon the extent to which fees have been scaled down below those shown by the normal curves originally established.

From time to time complaint has been raised as to the inadequacy of fees in defense construction and particularly against the requirement that a waiver of all claims under unfinished projects must be signed before a new contract can be awarded. However, leading members of the Society, long in responsible charge of large projects, are not only willing and anxious to take work under the later reduced schedules, but readily sign the waivers that are required. The fact is that it is our war, we feel a patriotic and proprietary interest in it, and our own members have been in charge of construction on more than 80% of the newly constructed Divisional Centers as well as many other projects of great magnitude and varying types.

Much credit goes to the Navy for pioneer legislation covering this "Cost-plus-a-fixed-fee" type of contract which greatly facilitated the entire emergency construction program; studies started as early as 1938 in connection with construction of the Pacific Island Bases finally became law by Act of Congress, April 25, 1939. Procedure under this particular type of contract is very fully described in a most interesting and valuable publication by the Bureau of Yards and Docks of the Navy Department.[4]

National Committee on Civilian Protection in War Time

Even in these hectic days not all of our Society's efforts are bent on the total ruin of the enemy, but some are directed to passive civilian defense, lest the enemy totally ruin us. I have reference to the work of our very efficient "National Committee on Civilian Protection in War Time." Work and organization of the main Committee are paralleled in most of our local sections and its activities are increasing in momentum.

It is recorded that when Walter D. Binger, M. Am. Soc. C. E., was first offered the chairmanship of this Committee he accepted "on condition that no further appointments would be made until close contact had been made with the War Department, after which he would select a committee whose personnel would embrace the type of man and mind best suited for civilian protection in war time." Subsequent events and the outstanding work of the Committee have proved that in making this stipulation Chairman Binger was wise beyond his generation.

There has been an orderly progress in the work of the Committee and in October of last year, at a meeting called by the Secretary of War within the War Department, arrangements were made for informal cooperation. Later, the War Department created the National Technological Civil Protection Committee consisting of one member each from eleven national societies, or organizations, together with a contact member from the War Department. In recognition of his effective pioneer work the Chairman of the Society's Committee was very properly appointed Chairman of the National Committee, which, it was announced, "would assist the War Department in technological matters relating to the collection and evaluation and dissemination of information of value in protection of civilians and civilian institutions in time of war."

I shall not at this time attempt to tell you of the rapidly growing numbers of our Society who are laying aside their civilian occupations and going directly into the armed forces of the country in all branches, and in all ranks. Only by a special roster or roll of honor can their services be recorded.

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[4] "Some Commentaries on 'Cost-Plus-a-Fixed-Fee' Contracts, with Particular Reference to United States Navy Contracts Under the Bureau of Yards and Docks," by William M. Smith, 1940.

THE ACTIVITIES OF OTHERS

So much for our own progress along three very distinct lines. I have also noted with interest the progress of our fellows, the American Society of Mechanical Engineers, with whose activities I have chanced to come into closer contact than with those of the other Founder Societies. Before June, 1940, that Society had a special Committee on National Defense, consisting up to that time of five members. Thereafter the Committee was expanded to twenty one, not including the Chairman and the Society's Secretary; in its three classes of members were four "Advisory Members" of whom three are also members of the Advisory Commission to the Council of National Defense and one is General Officer of the Army; two "Army and Navy Members" include a Captain from the Navy and a General from the Army; and the "General Committee" consists of fifteen civilian members, leaders in the profession.

It will be noted that this Committee of the American Society of Mechanical Engineers, containing as it does, two members of the Advisory Commission to the Council of National Defense, and three officers of high rank in the Army and Navy, is exceptionally well organized to secure the highest coordination between the military arms of the government and that Society.

Since September, 1940, also, great weight has been given on the programs of both National and Local Section meetings of the Mechanical Engineers to papers and discussions bearing directly on National Defense. In addition, the National Society has established and is maintaining strong contacts with certain research agencies, that should be of benefit both to the Society and the government.

These activities of our fellow engineers are of more than transient interest, since at the Baltimore meeting of the American Society of Civil Engineers after friendly negotiations with the Mechanical Engineers, and similar action by that Society, and after discussion with certain members of the Board of Directors of the American Institute of Electrical Engineers, our own Board of Direction provided for a local representative and joint headquarters in Washington and authorized cooperation with the other Founder Societies. It also authorized participation in a Joint Conference Committee consisting of the Presidents and Secretaries of the Founder Societies, to meet quarterly on matters of National Defense and on other matters to improve the solidarity of the engineering profession; certain modifications have been made in these arrangements since the time of the Spring Meeting, but it is to be hoped that they will be carried through.

From all of the foregoing it is clear that the engineer is enmeshed in "Total War." The civil engineer builds plants in which the mechanical engineer manufactures shells, tanks, and all other warlike paraphernalia, for which the mining engineer gives him steel and the electrical engineer power, and the greater perfection they evolve in their career of destruction, the sooner will they be able to return with their fellow men to peace and the pursuit of happiness, although whether the peace that follows this war will be the same calm peace, and the happiness, the same kind of happiness that we have known in years gone by, yet remains to be seen.

Gone are the sunny sleepy days of the Mission Fathers; gone are the constructive days of our own pioneer ancestors. All around we see ample signs of military and naval efficiency in the heavens above, on the earth beneath, and in the waters that surround the land.

We live in a tragic world of Total War; fewer are killed but more are enslaved. We cannot for a moment complain, however, that we live in an uninteresting age. Each day a new act in the world drama unfolds, and in each act the engineer will play to the full his part.

MEMOIRS OF DECEASED MEMBERS

ROBERT RIDGWAY, Past-President and Hon. M. Am. Soc. C. E.[1]

DIED DECEMBER 19, 1938

In the Borough of Brooklyn, N. Y. (at that time the small City of Brooklyn), with its cobblestone pavements, gas and oil lamps, curbstone pumps, horse-drawn vehicles, and only water transportation across the East River to the City of New York, Robert Ridgway was born on October 19, 1862. It was the middle of the Civil War period, in the first term of President Abraham Lincoln.

While returning by train from the ceremony of breaking ground for the starting of construction of the Chicago (Ill.) Subway, in the planning of which he had been consulted, he was stricken with what proved to be a fatal illness and died in a hospital at Fort Wayne, Ind., on December 19, 1938.

During his span of life he saw the cobblestones superseded by miles of modern pavements, and witnessed the birth of illumination by electricity and the harnessing of that illusive force to useful production and comfortable living. He played major parts in the elimination of the disease-spreading curbstone pump and in the betterment and modernization of transportation in Greater New York, of which his native city became an important borough.

Robert Ridgway was the next to the youngest child of a family of six boys and one girl. His home life was wholesome. His father, Joseph Skidmore Ridgway, was a lawyer. His mother, born Margaret Stephens, was devoted to her family and tried to instill in her children the true principles of correct living.

The formal education of Robert Ridgway began and ended in the public schools of the old City of Brooklyn. It began with his entrance into the primary class in 1870 in his eighth year and ended before his twelfth year in 1874 when his family removed to Greenbrook, Somerset County, N. J., near Sebring Mills, between Bound Brook and Dunellen. Here he continued his studies as best he could at home, using the textbooks of his elder brothers. He lived the healthy life of a country boy, which gave him an insight into many matters with which, in that day, the city boy had no way of becoming acquainted. He was fond of music and good reading, sang in church choirs, and read all good books on which he could lay his hands.

In the latter part of 1880 he moved with his family to a small place of a few acres near Cranford, N. J. Here his interest in music brought him acquaintance with the Rev. Walter C. Roberts and this acquaintance ripened into a lifelong friendship. Mr. Roberts was a graduate (Class of 1876) of Sheffield Scientific School, Yale University, in New Haven, Conn., where he had been trained as an architect. He had, however, made the ministry his lifework, and was then Rector of Trinity Church, in Cranford. A broad-minded, enthusiastic young man who understood humanity, he took a fancy to Robert Ridgway, whose lack of education he realized. So he stated tactfully that he would like to brush up on his own studies and would be glad to have Robert work along with him. He taught Robert Ridgway geometry, plane and spherical trigonometry, and encouraged him in a course of good reading. Mr. Roberts lived until 1930 and through all the years Robert Ridgway was his firm friend.

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[1] Memoir prepared by the late Otis E. Hovey, Hon. M. Am. Soc. C. E., John H. Myers, M. Am. Soc. C. E., and Harry D. Winsor, Assoc. M. Am. Soc. C. E.

The Ridgway family were inclined to the law, but Robert was more interested in things material that were governed by the laws of nature. Engineering appealed to him, but his lack of a technical training was discouraging. However, his youthful imagination was fired upon hearing that the Northern Pacific Railway Company had resumed construction from Bismarck, N. Dak., west, because he had always been interested in the West and hoped for an opportunity to work there. With this in mind, he took up with Mr. Roberts the study of elementary surveying.

Ambitious to do something and growing restless under his mode of life at Cranford, Robert finally won his father's consent to "Go West." Armed with letters of introduction and carrying what seemed to him a huge sum of money—$50—he left home in April, 1882. He was then under twenty years of age. Arriving at St. Paul, Minn., he presented a letter of introduction to General Herman Haupt, general manager of the Northern Pacific Railway Company, who referred him to the late General Adna A. Anderson,[2] M. Am. Soc. C. E., chief engineer, with headquarters at Brainerd, Minn., on the Mississippi River. Here, on Saturday, April 29, 1882, he was received by a direct and gruff assistant and was given no assurance of a job. However, on the following Monday, May 1, 1882, he learned that he was to go to the end of the line in Montana and was placed upon the pay roll of the railroad, thus beginning his first engineering work. From Brainerd, he journeyed westward by train through Fargo, then in Dakota Territory, through Bismarck where the train was ferried over the Missouri River, and on to Glendive, in Montana Territory. From Glendive the train proceeded southwest up the valley of the Yellowstone River to Miles City, Mont., at that time the end of the operated track. He traveled in the company of Daniel S. Hastings, a locating engineer from whom he learned that he was to be a member of a surveying party that was to locate a line from near Billings on the Yellowstone River to Fort Benton, Mont., at the head of navigation on the Missouri River, a distance of about 250 miles. From Miles City to Billings the journey was continued first by freight car and then by wagon. Ultimately, the party took to tents and, escorted by a detachment of U. S. Cavalry for protection against Indians, carried the survey from the Yellowstone River to the Missouri River in about three months. The surveying party then worked back over the line in an effort to improve it and reached the Yellowstone River near Forsyth Station, Mont. From here, Mr. Ridgway returned to Billings by train, as the operation of the Northern Pacific Railway had by that time (November 1, 1882) been extended. After other surveys in the neighborhood of Billings, he was assigned to office work until after the end of the year 1882. At night, while in camp, he continued to read and study and was thus acquiring an all-round education.

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[2] For memoir, see *Proceedings*, Am. Soc. C. E., Vol. XV (1889), p. 166.

In January, 1883, orders were received to revise and improve a railroad location which had been made between Superior, Wis., and Ashland, Wis., 60 miles east. This survey was made in the dead of winter through forest, the men being housed in tents and using snowshoes in their work. During this trip he continued his reading and studies. A candle held in a split stick stuck in the ground at the head of his bed furnished the light. The survey reached Ashland about April 1, 1883, despite the fact that once the usual supply of food failed to reach the party on time. During the summer and fall that followed, Mr. Ridgway had his first experience on construction work as a rodman on this piece of railroad. He lived in a log house where, on October 19, 1883, he celebrated his twenty-first birthday by receiving the congratulations of his associates.

In December the work closed down and he reached Cranford in time to enjoy the Christmas holidays of 1883 with his family. He hoped to return to the West but no opportunity appeared; idleness began to pall on him; he again became conscious of his lack of a technical education; and he became discouraged. However, in July, 1884, a message was received to report as leveler at Superior, and his spirits again rose. From that time until his death, it may be said he was never without employment. He took up construction work on the railroad between Superior and Ashland and was again happy in a log house. This assignment was to last only three weeks, however, because a message then came asking him to report as a leveler to the chief engineer of the Aqueduct Commission in New York City.

With great reluctance young Ridgway left the West and began work at Tarrytown, N. Y., in August, 1884, on the building of the second aqueduct for carrying the water of the Croton watershed to the City of New York. His salary was $100 a month. He reported to the late Henry S. Craven,[3] M. Am. Soc. C. E., who was in charge of all of the field work of the Aqueduct Commission. The chief engineer was B. S. Church. On that work also were the late Charles S. Gowen,[4] M. Am. Soc. C. E., under whom Mr. Ridgway worked for many years, and a brother of Henry S. Craven, the late Alfred Craven,[5] M. Am. Soc. C. E., with whom Mr. Ridgway was destined to be associated later in the construction of the rapid transit lines of the City of New York.

The alinement of the "new" Croton Aqueduct, as it was termed, had been fixed between the Harlem River and the "old" Croton Dam. With the other engineers, Mr. Ridgway worked on perfecting the alinement, and on accurately chaining, establishing bench marks along, and carefully monumenting the line so that it could be used readily in constructing the aqueduct, which was to be in tunnel. This, following after his experience on railroad work, no doubt perfected his technique in surveying, of which he became a master. Similarly, the construction that came later gave him a great opportunity to perfect himself in that art, and he embraced every opportunity to learn. His knowledge of engineering was based on a sure foundation because he learned largely by observation in the hard school of experience.

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[3] For memoir, see *Proceedings*, Am. Soc. C. E., Vol. XVI (1890), p. 216.

[4] For memoir, see *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 1671.

[5] For memoir, *loc. cit.*, Vol. 94 (1930), p. 1506.

He was associated with the building of a part of the aqueduct adjacent to Croton Lake and the gate-house on its shore, which controlled the flow of water into the aqueduct. The aqueduct tunnel was driven from shafts about a mile apart, and the first real work done was the sinking of the shafts, Nos. 0 to 4, inclusive, on the 4.5-mile division to which Mr. Ridgway was assigned as a leveler. Mr. Gowen was the division engineer. Shafts Nos. 1 to 4 were vertical shafts; and shaft zero was inclined. Lines were projected into the tunnels from piano wires properly set on the surface and carrying 24-lb weights hanging in barrels of water to prevent swaying. Levels were transferred down the shafts by means of a hanging steel tape with a 12-lb pull. These operations were repeated many times until Mr. Gowen was satisfied, his maxim being "always consider what you have done is wrong until you prove it is right." About a year was consumed in sinking these shafts.

In 1885 Robert Ridgway became a transitman at $1,500 per annum and in 1886 was promoted to the grade of assistant engineer at $1,800. The work was divided and he was put in charge of the upper part, which included shafts Nos. 0 and 1, the intervening aqueduct, and the gate-house.

Time was when the expression "an honest contractor" was a byword. He was cited as "a rare bird" and the intimation was that, as a rule, the species was not honest. The fact was that they were no doubt as honest as the general run of mankind of their day and generation; but, engaged as they often were on the construction of large public works, their shortcomings were sometimes much in the public eye. Such was the case on the new Croton Aqueduct. Certain of the work was defective, the engineers refused to pay for it, and, as Mr. Ridgway wrote years afterward, "this led to a hullabaloo." A legislative investigation followed, the Aqueduct Commission was reorganized, a new chief engineer, the late Alphonse Fteley,[6] Past-President, Am. Soc. C. E., was appointed, and Messrs. Gowen and Craven with others, among them Mr. Ridgway, continued on the work. The defective work was corrected. The gate-house, which was a massive and complicated masonry structure, was completed under the direction of Mr. Ridgway, except for its superstructure.

The new Croton Aqueduct was placed in service in 1890. In that year Mr. Ridgway was sent to Purdy's Station, N. Y., to take charge as assistant engineer of the construction of Titicus Dam and of the reservoir officially known as Reservoir M of the Croton System. The purpose of the dam was to develop the Croton watershed more fully. It was of earth and masonry with an overfall in the masonry part 200 ft long. The earth parts had masonry core walls. Mr. Gowen was the division engineer at the beginning but was succeeded by Alfred Craven.

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[6] For memoir, see *Transactions*, Am. Soc. C. E., Vol. LIV, June, 1905, p. 509.

In the spring of 1895, upon the completion of the dam, Mr. Ridgway was transferred to the Borough of the Bronx, New York City, to supervise the construction of the Jerome Park Reservoir. Its site was on high land between Jerome and Sedgwick avenues north of Kingsbridge Road and it derived its name from the celebrated race track formerly located there. Again Mr. Craven was the division engineer. The Jerome Park work was principally an excavation job, involving approximately 7,000,000 cu yd. Two basins, an east and a west, were provided. The original Croton Aqueduct traversed the site, and its equivalent was to be constructed in the wall which separated the two basins. Dams or dikes that were to form the perimeter of the reservoir were of earth with rubble masonry cores. A central gate-house controlled the flow from the old and new aqueducts. The new aqueduct lay beneath in deep tunnel and a connecting conduit was planned. Other gate-houses controlled the flow from the basins to the city mains.

After the work had been in progress some time, efforts were made to change the plans by substituting masonry walls for earth embankments with masonry cores. This was opposed by Mr. Fteley; but after his resignation the new chief, the late William R. Hill,[7] M. Am. Soc. C. E., substituted masonry along Sedgwick Avenue. There was considerable controversy over the matter and Mr. Craven, the division engineer, who was not in sympathy with the changes, was transferred to another division of the work. Thereupon he sought and secured the position of division engineer with the Rapid Transit Railroad Commission which was about to undertake the construction of the first subway in the City of New York. He began his new duties on May 1, 1900.

Mr. Ridgway likewise secured employment on the same work, of which the late William Barclay Parsons,[8] Hon. M. Am. Soc. C. E., was chief engineer. On May 9, 1900, he became the principal assistant to Alfred Craven on what was known as the Second Division—that is, the subway under 42d Street from Park Avenue to Broadway and under Broadway from 42d Street to 104th Street. There were four tracks, two for local and two for express service. The subway engineers entered into the work with zest. They were a fine body of men and were well organized and directed, largely due to the foresight of the late George S. Rice,[9] M. Am. Soc. C. E., the deputy chief engineer, who selected most of them. By no means the least zealous of them was Robert Ridgway. Because of his experience in construction work during the previous eighteen years, coupled with his disposition and thoroughness, he proved a model leader for the younger men who served under him. He never drove them, but he gently set a pace which they naturally felt it their duty to follow. He carried out the details of the general procedure laid down by Mr. Craven.

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[7] For memoir, see *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1687.

[8] For memoir, *loc. cit.*, Vol. 98 (1933), p. 1485.

[9] For memoir, *loc. cit.*, Vol. LXXXV (1922), p. 1713.

The subway structure in itself is a simple one—a bent of columns and roof beams every 5 ft, with wall and roof arches between—but the building of such a structure under the highways, and below, and often close to the foundations of buildings in the largest city on earth, involving as it does the maintenance of traffic, the re-routing and maintenance of a maze of subsurface structures, and withal the safety of citizens and workmen alike, makes it a fit task for the talented engineer. Robert Ridgway measured up to this task. Day after day he tramped over the work, on the surface or through its excavations and partly completed structures. Now and then his men received a gentle reminder of their shortcomings, but they were never "dressed down" and orders were never given over their heads.

Before construction began he interested himself in establishing the base lines. It was done with the idea not only of using them as something on which to hang a survey of existing conditions, but also of providing a ready reference from which to lay out the work. The construction drawings were referred to these base lines. The center line of the structure was fixed in its final position and temporarily marked, notwithstanding trolley cars and truck drivers. On 42d Street this work started at daybreak. A profile was made and cross sections were taken from building line to building line and into the side streets at intersections. At the same time, parallel base lines, well marked every 50 ft, were run on each sidewalk where they were less likely to be disturbed. On curves, shorter chords were used on one base line and longer ones on the other so that a line from one to the other passed radially through the proper station on the center line.

He compiled a field book which was a model. On its right-hand page were a map and profile of the line showing base-line stations, stations of points of curvature (P.C.) and points of tangency (P.T.) of horizontal curves and their radii, street-surface elevations at intersecting streets, rates of grade of base of rail, governing elevations where grades changed, the P.C., P.I. (point of intersection), and P.T. of vertical curves, and their rate of change. On the left-hand page was a table showing stations every 50 ft, and at the horizontal and vertical P.C. and P.T., elevation of base of rail at all these points, total deflections to each station on all curves, and the deflection and versed sine for standard chord. The original tracings were stored in the Division Office safe and the several books were of bound white prints. In the back of the book were tables of bench marks, corrections for vertical curves, and cross sections of the standard structure with flaps to which other sheets of information could be pasted.

So complete was the information that a party could lay out from it any part of the work except the depots, which were complete structures in themselves. The book was a timesaver and a guard against error. It is evidence of his superb command of the minutiæ of his profession. This work behind him, he gave his attention to the broader phases of the construction work. Probably he worked harder than any man on the division.

Notwithstanding his modesty, his reputation apparently reached the chief engineer's office, and his ability became known to Chief William Barclay

Parsons who, when the construction of the loop at the Battery in Manhattan, the tunnel under the East River, and the subway along Joralemon and Fulton streets and Flatbush Avenue in Brooklyn, was to be undertaken, selected him as the division engineer. With regret, he left Mr. Craven for whom he had great admiration; but he was again to serve under him when Mr. Craven became chief engineer of the subway work. The new division was designated as the Fifth and Mr. Ridgway took charge on March 23, 1903.

The outstanding feature of this work was the tunnel under the East River from the Battery to Brooklyn. This, the first transit tunnel to be built under the East River, consisted of two cast-iron circular tubes driven by the shield method. To a considerable degree it was pioneer work. Something like thirty years later Mr. Ridgway wrote "the art of compressed shield tunneling has been brought to a high state of development through experience gained under the waters of New York harbor." The work was beset with many difficulties, among them an inadequate compressor plant, cast-iron rings that were too light, and a severe fire in some timbering. However, it was completed and is today a useful link in the transportation system of New York City. The Battery Loop was a complicated structure built of reinforced concrete in an excavation where the water level was affected by the rise and fall of the tides. The subway work in Brooklyn was less complicated.

All this work Mr. Ridgway supervised with skill and diligence. In 1905, the late J. Waldo Smith,[10] Hon. M. Am. Soc. C. E., a keen judge of men and a great executive, was beginning the work of building the great Catskill Aqueduct and was assembling an engineer corps for that purpose. He offered Mr. Ridgway the position of division engineer on this work. It was intimated that he would be given the position of department engineer when the permanent organization was effected. This offer, coming from J. Waldo Smith, was a tribute to the ability and character of Robert Ridgway. He accepted the offer and took up his new duties on September 1, 1905.

He took part in the studies and surveys for the plans for the northerly 60 miles of the Catskill Aqueduct and later supervised its construction. His jurisdiction was known as the Northern Aqueduct Department and extended southerly from the Ashokan Reservoir to Hunter's Brook, 5 miles or more southeast of Peekskill, N. Y. The work was diversified; two thirds of it was cut-and-cover masonry aqueduct following approximately the contour of the land. Minor depressions were crossed by steel-pipe siphons. The remainder was in tunnel, piercing mountains or their spurs or passing under the deeper valleys. In the former case the tunnels often followed the hydraulic grade, while in the latter they were below the hydraulic grade and under considerable pressure, notably the tunnel which carried the aqueduct beneath the Hudson River. The conception and construction of the dams, aqueducts, and reservoirs for impounding Catskill water and conveying it to the City of New York was a monumental work. Robert Ridgway had a major part in it and played his rôle well. He was engaged on it from his forty-third to his forty-ninth year.

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[10] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 101 (1936), p. 1502.

In the fall of 1911, the work was well along toward completion, and its major problems had been solved. About this time the rapid transit work of the City of New York, which had not been specially active while Mr. Ridgway was engaged on the construction of the Catskill Aqueduct, began to assume large proportions. An attempt was being made by the authorities in charge, the State Public Service Commission for the First District, to develop a so-called "Triboro System" that would be independent of any of the existing railroads. The chief engineer of the Commission, Mr. Ridgway's old superior on aqueduct and rapid transit work, Alfred Craven, offered him the position of engineer of subway construction, which position was the equivalent of deputy chief engineer. With the blessing of J. Waldo Smith, Mr. Ridgway accepted. January 15, 1912, saw him in this new position.

The Triboro System never came into being. Instead, after much discussion and many setbacks, the so-called "Dual Contracts" between the City of New York and the Interboro Rapid Transit Company and the Brooklyn-Manhattan Transit Company, respectively, were drafted and signed. They provided for large extensions of the roads operated by these companies. The Brooklyn railroads came into Manhattan and Queens and those in Manhattan were extended into Brooklyn and Queens; and the original four-track subway in Manhattan became two railroads of four tracks each. It was a colossal job, much more complicated than the first subway. After the United States entered the first World War, its effects on the work began to be felt in scarcity and high cost of labor and material. Nevertheless, the railroads were completed under the supervision of a large corps of engineers directed by Mr. Ridgway, and on August 1, 1918, the two four-track railroads in Manhattan began operation. Although there were several changes in the administration of New York City transit affairs following the building of the dual system, the engineer corps continued its work. The governing body in the year 1921 was known as the Transit Commission, and Robert Ridgway was made its chief engineer on May 9 of that year, exactly twenty-one years after he came on the work as an assistant engineer in 1900.

Further changes brought into existence, in 1924, the Board of Transportation, and Mr. Ridgway continued with this Board also as its chief engineer. It made a major addition to the transit facilities of New York by constructing and operating an "Independent System" which enters all the boroughs of the city except Richmond. Even more complicated than the dual system, this work went forward with no World War to delay it. It necessitated the rerouting of subsurface structures and the underpinning of buildings as the previous work had; and, in addition, it encountered masonry aqueducts, groups of large water mains, elevated railroads, monuments, and existing subways. All these interfering structures were remodeled to permit the construction of the subway. Parts of the railroad were in subaqueous tunnels. It was a vast undertaking, possibly the largest project ever undertaken by any municipality.

Robert Ridgway reached the age of seventy soon after the operation of the Independent System began, and he applied for retirement to take place November 1, 1932; but owing to the sudden death of Col. John R. Slattery,[11]

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[11] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 99 (1934), p. 1511.

M. Am. Soc. C. E., who was to succeed him, he continued as consulting engineer until December 16, 1933, thus terminating a continuous service with the City of New York of more than forty-nine years, since August, 1884. During this period he had been responsible for engineering construction totaling well over a billion dollars. At an informal luncheon tendered him the previous day by his staff, one of them said:

> "Mr. Ridgway, our sentiments today are a one-to-one mixture of regret and pleasure; regret that we have come to the parting of the ways but on the other hand, pleasure in knowing that you are to be relieved of the burdens which apparently accompany a job of this sort and which for years you have so patiently borne. You will always have the esteem and affection of each one of us. We wish you joy and happiness along whatever paths your footsteps may follow in the future."

This voiced the feelings of his staff toward their chief.

It is not surprising that the abilities of a man who had served the great City of New York on major construction work for nearly half a century should be sought by others who had similar problems to solve. Thus, with William Barclay Parsons and Bion J. Arnold, M. Am. Soc. C. E., he became a member of the Chicago Traction and Subway Commission which made a study of the Chicago transit problem in 1916–1917. His services were again sought by the City of Chicago thirteen years later, and he had a part in preparing plans for a Chicago Subway. Other consulting work included the San Francisco-Oakland Bay Bridge in 1927, Colorado River Board (Hoover Dam) in 1928, Midtown Tunnel, New York City, in 1933, and the San Francisco Subway in 1935.

In October, 1929, Robert Ridgway went to Japan to attend the World Engineering Congress. With others he was commissioned by President Herbert Hoover, Hon. M. Am. Soc. C. E., to represent the United States. He also represented the Lehigh and Harvard engineering societies. Noted for his hospitality, it was his habit to entertain those who came to visit the engineering work he supervised for the City of New York. Among such visitors were many Japanese engineers who, when the opportunity afforded by his presence at this engineering congress presented itself, did their utmost to reciprocate his hospitality. He spent three months in the Orient. The delegates were received and entertained by the Japanese Government and the English societies, and Mr. and Mrs. Ridgway enjoyed the hospitality of Japanese engineers who had visited Mr. Ridgway's work in the United States. The Emperor of Japan conferred on him the Order of the Rising Sun.

Institutions of higher education in the United States recognized Mr. Ridgway's ability and conferred degrees upon him. New York University, in New York City, honored him with two—Master of Science in Civil Engineering in 1915 and Civil Engineer in 1919. Lehigh University, in Bethlehem, Pa., gave him a Doctor of Engineering in 1929, and Harvard University, in Cambridge, Mass., bestowed upon him its Master of Arts in 1925. In 1933 he received a Doctor of Engineering from the Polytechnic Institute of Brooklyn.

Mr. Ridgway was twice married. His first wife was Lillie A. Littell to whom he was married on May 10, 1888. She died on March 12, 1927. She shared his years on the new Croton Aqueduct work, the first subway work, the Catskill Water Supply, and the dual system of subways. On September 15, 1928, he married Isabel L. Law who survives him. The ceremony was performed by the Episcopal Clergyman from Canterbury, New Brunswick, Canada, assisted by the Rev. Walter C. Roberts who had officiated at his first marriage. This illustrates the sincerity of Robert Ridgway's friendship. It was Mr. Roberts who had befriended the boy Robert Ridgway and had schooled him in mathematics. They were lifelong friends. To Isabel Ridgway he dictated in spare moments his recollections. After his death she had them printed and sent copies to his many friends. The book "Robert Ridgway"[12] is wholesome reading, especially for the young and ambitious engineer.

Two days before Christmas, 1938, funeral services were held for Robert Ridgway in the Church of the Transfiguration (The Little Church Around the Corner), New York City. The rites were those of the Protestant Episcopal Church, simple and impressive, and, as he would have wished it, without eulogy. The edifice was thronged with his friends who came to pay their last tribute. Their number testified to the esteem in which he was held more eloquently than any eulogy.

Robert Ridgway was a member of the Institution of Civil Engineers (Great Britain); Past-President and Honorary Member of the Harvard Engineering Society; charter member and Past-President, Municipal Engineers of the City of New York; a member of the Franklin Institute, the American Association for the Advancement of Science, the American Society for Municipal Improvements, the American Association of Engineers, the Society of American Military Engineers, and the New England Water Works Association.

Mr. Ridgway was elected a Junior of the American Society of Civil Engineers on February 1, 1888, a Member on June 3, 1903, and an Honorary Member on October 2, 1934. He was presented for the latter award by George S. Davison, Past-President and Hon. M. Am. Soc. C. E., at the Annual Meeting on January 16, 1935. Mr. Ridgway served the Society as Director from 1911 to 1913, as Vice-President in 1922 and 1923, and as President in 1925. He was for some time the Secretary-Treasurer of the Alfred Noble Prize Committee, and gave freely of his time and experience to a number of other Society committees.

Any one reading the life of Robert Ridgway would be duly impressed by the scope of his engineering accomplishments; but this perusal would give no clue to the wonderful character that was his. However, his character was soon evident to those whose contacts with him were only casual; and those whose privilege it was to know him more intimately found him not only a great engineer but a patriotic and conservative American, a broad-minded, progressive, tolerant, and friendly man. Although with scant formal schooling, he was a master of the technique of his profession and an old-school gentleman of broad reading and culture, well versed in what the moderns call the humanities. One should not get the impression that he looked down on formal education. Quite the contrary. He cherished the honorary degrees that were conferred upon him and delighted in addressing gatherings of engineering students to whom, in addition to urging them to pursue faithfully their engineering studies, he preached patriotism and good citizenship.

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[12] "Robert Ridgway," autobiography, published privately, New York, N. Y., 1940.

His friends were legion and he took great delight in their company. With no idea of currying favor, he was a friend alike to great and small. He was a man of no hobbies unless work and people may be called such. Once with one of his division engineers he went to inspect some of the New York City subway construction. The division engineer was glad of the opportunity to escort his chief over the work because the multitude of duties in the chief's office rarely permitted this. The inspection, however, proved to be a sort of reception. He shook hands with the stone mason laying an ashlar retaining wall; he had known him in the new Croton Aqueduct days. He chatted with the contractor's superintendent whom he had known on other subway work. Several of the members of the engineer corps of long service shared his greetings. He spoke to a laborer—the son of an engineer—who had a "vacation job." This well illustrates the catholicity of his friendships.

These friendships extended far beyond the confines of even a huge city. By the thousand, all over the country, he was privileged to call people "friends." His travels, particularly those as an officer of the Society, only multiplied the number. There was hardly a trip, on Society or other business, that he did not make it a point, even to going far out of his way, of hunting up old cronies or casual acquaintances. His remarkable memory for names and for personages, high or low, only accentuated this wonderful genius for friendship.

Incidents could be multiplied to show the true worth of his character. One will suffice, as it indirectly concerns the Society. It was on an inspection trip, incident to an Annual Meeting in New York. The bus was only moderately full. As Mr. Ridgway was in deep conversation with a companion, he noted an eminent member of the Society, sitting alone, and he also noted a young engineering friend, likewise disengaged. Impelled by the real courtesy that was ingrained, he approached his young friend and suggested that "Past-President ——— seems to be alone and would doubtless relish a chat." The idea was put into practice with good results, certainly as far as the young man was concerned; but the prominent engineer, also a friend of Mr. Ridgway, had not the least idea that he had been the object of any one's solicitude. Robert Ridgway was just like that.

His designs for engineering structures were not mere hurdles over which contractors were to find ways of leaping. They were rather the setting of necessary tasks that he knew, from his long and intimate contact with construction work, could be accomplished by careful and thorough work on the part of competent contractors. He was always ready to give ear to, to digest, and to use in part or in whole, the ideas and experience of the great and the humble alike, and never did he fail to give credit to whom credit was due. Rising above and seeing beyond bending moments, column loads, and soil mechanics, he attacked his problems with a breadth of vision that resulted in wise solutions.

Like some of the men whose graves he saw in the cemeteries of the pioneer towns of the West, and who were said to have "died with their boots on," in his gentle and modest way he worked to the day of his death. He lived a long, honorable, and useful life. He had a part in planning and building many structures that are a lasting benefit to man and by his fair and courteous dealings with his fellow men he made hosts of friends. The structures that he reared and the friendships that he made are lasting monuments to this noble man.

RAYMOND FRANCIS ALMIRALL, M. Am. Soc. C. E.[1]

DIED MAY 18, 1939

Raymond Francis Almirall, architect, was born in Brooklyn, N. Y., on March 1, 1869, the son of Joseph Janer and Ida (Gunn) Almirall. His father, a member of an old and noble Catalonian family, came to the United States from Spain as a young man, developed a successful importing business, and later was head of the West Indian Oil Refining Company. On his mother's side he was descended from early English colonists in Maryland.

Raymond Almirall received his preparatory education at the Polytechnic Institute of Brooklyn and was graduated with the degree of Bachelor of Science in Architecture at Cornell University, at Ithaca, N. Y., in 1891. From 1892 to 1896 he studied architecture under Victor Laloux at the École des Beaux Arts in Paris, France.

After his return to the United States he practiced architecture in New York, N. Y. Among the numerous well-known buildings designed by him are Fordham and Gouverneur hospitals, New York City; Sea View hospital, Staten Island; a sanitarium, East Moriches, Long Island; the Central Building and Pacific, Prospect, Bushwick, and Eastern Parkway branches of the Brooklyn Public Library; Kingston (N. Y.) Public Library; St. Michael's Church and the Church of the Nativity, Brooklyn; the Roman Catholic churches at Glen Cove, Oyster Bay, and Sea Cliff, Long Island, and Ravenswood, Long Island City, N. Y.; Episcopal Church, Franklin, Pa.; the Novitiate of the Little Sisters of the Poor, Queens, Long Island; Brooklyn, Bronx, Manhattan, and Jamaica homes for the aged, Little Sisters of the Poor; agricultural school of the New York Catholic Protectory, Somers Center, N. Y.; New Jersey state reformatory, Rahway, N. J.; farm colony, Randall's Island, New York City; Emigrant Industrial Savings Bank building, New York City; Binghamton (N. Y.) Savings Bank; municipal lodging house, New York City, Brooklyn public bath; the Shotter residence, Savannah, Ga.; Alexander residence, Elizabeth, N. J.; and the mortuary chapel, Calvary Cemetery, Long Island City.

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[1] Memoir prepared by Joseph Janer Almirall, New York, N. Y.

For some years he was a member and secretary of the advisory commission in charge of erecting the Carnegie libraries. Mr. Almirall was greatly interested in civic affairs. In 1900 he was appointed a member of the tenement house commission by Theodore Roosevelt, then governor of New York. This commission was responsible for many of the tenement house laws that later went into effect. From 1919 to 1921 he was foreman of an extraordinary grand jury which was called to investigate an alleged conspiracy on the part of officials of the Interborough Rapid Transit Company and the brotherhood of the company's employees to raise the subway fare.

In 1924 he became the voluntary and unofficial representative in Paris of Welles Bosworth, architect in charge of the restorations of the Palaces of Versailles and the Trianon for John D. Rockefeller, Jr.; and for this work was made a chevalier of the Legion of Honor by the French government on July 25, 1928.

Mr. Almirall was not only an artist of outstanding creative talent, whose conception and expression of beauty and form left their permanent mark in the world of architecture, but he was also an idealist who, when necessary, unhesitatingly sacrificed his own personal advantage for the sake of his convictions. No matter how difficult of attainment the ends he envisioned in personal, civic, or professional undertakings, his strong will, his perseverance, and his absolute integrity always sustained him in their accomplishment.

Artist, leader, philosopher, he was beloved and admired by his associates and by all who came to him for help and guidance. The welfare and happiness of humanity were the inspiration of many of the buildings which his love of architecture and his genius produced.

Mr. Almirall was a member of the American Institute of Architects, Society of Beaux Arts Architects, Chi Psi Fraternity, Racquet and Tennis Club of New York City, Meadow Brook Club of Westbury, N. Y., and Piping Rock Club, Locust Valley, N. Y., Metropolitan Club, Washington, D. C., and the Interallie, Paris, France. In religion he was a Roman Catholic; in politics a non-partisan.

He was married in New Orleans, La., on June 2, 1897, to Marguerite Allain Cusachs, daughter of Pierre Leon Cusachs, a chemist of that city, and they had three children: Joseph Janer, Francis Cusachs and Raymonde Cusachs (Mrs. Don Morrison Kelley).

Mr. Almirall was elected a Member of the American Society of Civil Engineers on October 5, 1904.

CARL ASHLEY, M. Am. Soc. C. E.[1]

DIED FEBRUARY 3, 1941

Carl Ashley was born in Honeoye, N. Y., on August 17, 1878, the son of Frederick L. and Fannie (Briggs) Ashley.

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[1] Memoir prepared by Joel D. Justin, M. Am. Soc. C. E.

His technical education was obtained at the University of Nebraska, Lincoln, Nebr., and at Cornell University, Ithaca, N. Y. He was graduated with the degree of Civil Engineer in June, 1905.

Mr. Ashley began his professional career in Albany, N. Y., in 1905, with the New York State Barge Canal. From 1911 to 1914 he was the engineer in charge of construction and had responsible charge of layout and inspection. He designed locks, dams, retaining walls, temporary bridges, cofferdams, plant buildings, etc.

From 1914 to 1917 he was associated with the Gillespie Contracting Company and East Jersey Pipe Corporation, at Paterson, N. J. He was construction engineer on the Third Tracking of the Manhattan Elevated Railway having charge of all engineering for the Gillespie Company.

In July, 1917, Mr. Ashley enlisted in the United States Army and was appointed Captain in the Engineer Corps. Later he was commissioned a Major with the American Expeditionary Forces Transportation Corps in France. He had charge of Inland Water Transport in the Paris District until November, 1919. On his return to the United States, Major Ashley again entered the employ of the Gillespie Contracting Company.

In 1924 Major Ashley was employed by the American Construction Company of Cleveland, Ohio, as chief engineer and later vice-president on marine and heavy foundation construction. In 1933 he resigned and associated himself with James H. Small and Company, of Cleveland, as vice-president, in designing and building bridges and other structures.

From 1935 to 1937, Major Ashley was project engineer with the National Park Service at Birdsboro, Pa. His work covered highway construction, building of wood camp buildings, and a small dam.

In February, 1937, he became an associate of the writer, his former colleague and friend, Joel D. Justin, M. Am. Soc. C. E., in Philadelphia, Pa., where he remained until his death. He was proficient in both the design and construction of hydraulic structures and, as an engineer in charge, successfully executed important engineering projects. He had unusual ability for mathematical analyses. His intimate knowledge of both Spanish and French was of material value in connection with his technical work.

His untimely death was a severe shock to his relatives, friends, and associates. Carl Ashley never married and is survived by a sister, Evaline M. Ashley. He was a man of the highest character and was one of those rare people who invariably considered the rights and preferences of others before he considered his own. As a consequence he was beloved by all who knew him. Throughout his life he continued to operate his ancestral farm near Honeoye, and spent most of his vacation periods in that vicinity.

Mr. Ashley was elected an Associate Member of the American Society of Civil Engineers on December 2, 1914, and a Member on March 5, 1928.

TIMOTHY RALPH ATKINSON, M. Am. Soc. C. E.[1]

Died January 27, 1937

Timothy Ralph Atkinson was born in North Anson, Me., on February 9, 1868. He was the son of Joseph and Satira (Wells) Atkinson. The family was of Yankee-English-Scotch-Irish extraction, originally coming from the South and settling in Maine in near-Revolutionary days.

Mr. Atkinson attended Anson Academy, and then enrolled in the civil engineering course at the University of Maine, at Orono, Me. His scholarship was good, and he probably would have graduated with the Class of 1892, except that near the close of his junior year he took summer employment with a survey party on location or construction of branches of the Bangor and Aroostock Railroad (which ultimately became part of the Boston and Maine Railroad). He made such rapid and profitable advancement that the temptation to defer his formal education for a time was irresistible; thus he never received his degree.

He was occupied thus on railroad construction, maintenance, and improvement, making his home at Boston, Mass., until 1899. He then moved to Fargo, N. Dak., to become general manager of a flax-processing mill that was being tentatively promoted. The process finally was found to be unprofitable, and in a few years the mill was closed. Thereafter, he worked in the office of the city engineer of Fargo, and was himself the city engineer in 1904 and 1905.

At that early period in the growth of North Dakota, accurate surveying and civil engineering were mostly in the future tense, except for townsite layouts, simple forms of water works and sewerage systems, and simple bridges. There was no state highway department, and scarcely any other general directing agency for that or any other form of state work; but the national Reclamation Act had just been passed, and the citizens of the state wished to benefit by it if practicable. Early in 1904 they petitioned the governor of the state to appoint for the first time a state engineer to investigate such matters, and especially to initiate irrigation work in the state in regular form if found advisable.

Accordingly, the writer, as civil engineer of the state university faculty, was appointed to that task, and commenced at once the requisite surveys, researches, and formulation of purposes. In particular, he drew up for submission to the next session of the legislature a complete law, making the position of state engineer a regular permanent state office, including other appropriate fields as well as irrigation, with appointment for a four-year term. At the close of February, 1905, this was passed by the legislature in exactly the recommended form. To succeed himself as the permanent full-time occupant of the office the writer then nominated A. Lincoln Fellows of Colorado (as being the most competent obtainable man for directing irrigation development, which was the most important part of the immediate work), and asked him (after his appointment and arrival in North Dakota early in April to begin work) to take Timothy R. Atkinson as his chief assistant, or deputy state engineer. Mr. Atkinson, energetic and competent, was considered eminently suitable, except that he had not yet had any personal experience in irrigation work. Thus Mr. Atkinson began work in the state engineer's office on June 19, 1905.

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[1] Memoir prepared by E. F. Chandler, M. Am. Soc. C. E., with the assistance of Myron H. Atkinson, City Auditor, Bismarck, N. Dak.

Mr. Fellows resigned on July 1, 1907, to accept more responsible work in Colorado. That same day Mr. Atkinson, being the best possible candidate, was appointed for the unexpired part of the term; several times he was reappointed at the close of successive terms.

Under his direction, the work of that recently established office rapidly developed and enlarged, so as to include (besides irrigation) drainage, flood prevention, mine inspection, highways, bridges, some forms of municipal improvement, and various other fields. Meanwhile, much work that was not specifically included in the official scope of his office was offered to, and urged upon, him continuously by corporations, individuals, counties, or municipalities. The clients hoped that this work could be done under his supervision or, with his approval, at the hands of his assistants or others employed by him. Thus, Mr. Atkinson finally concluded that the field before him would be larger and more enjoyable if he were not holding the state office but were merely a private consulting engineer; so he resigned as state engineer on July 1, 1913.

Thereafter the work in which he and his force were engaged became really more widespread and important. If any one had been asked to name the half dozen leading civil engineers of the state during the following twenty or thirty years, Mr. Atkinson certainly would have been included in the group. His work was always progressive, but in every way correct and reliable; he did not urge over-development or needless expenditure of funds; and he was never involved in political disputes or other disagreements that impaired the universal confidence in the justice of his recommendations.

In addition to his private practice, he became the official city engineer of Bismarck, N. Dak., in 1907, and the county surveyor of Burleigh County, North Dakota, in 1914. With the exception of one of the terms as county surveyor, he held these positions continuously until his death, which occurred rather suddenly on a day when he was comfortably at work in his office.

Mr. Atkinson was liked and respected by all his associates and subordinates. Special mention should be made of one of his characteristic traits—his helpfulness to his employees. Many of his temporary assistants, rodmen, and draftsmen were quite young, perhaps merely inexperienced high-school graduates, and afterward they always described their work under him as an excellent education. He compelled each one to understand completely every step of any work done under his direction, instead of letting him merely follow orders blindly. Thus, if later (following Mr. Atkinson's encouragement) the young man went to college, he was almost certain to make a high scholastic and practical record, and after graduation progressed much faster than the usual civil engineering graduate. With this fortunate start in their training, many of

Mr. Atkinson's former young assistants scattered over North America now hold high positions as state, federal, or corporation engineers.

His chief interests outside of his profession were his family, the study of history (particularly North Dakota history), and gardening; his large gardens in the outskirts of Bismarck, partly irrigated, were his pride. For a time he also operated somewhat larger irrigation projects in the region. He was an enthusiastic member of the Bismarck Rotary Club, and of the Bismarck Association of Commerce, and was in many other civic activities. He was a member of St. George's Episcopal Church in Bismarck, a member of its vestry, and for many years rector's warden.

He was married to Emma Hilton at Wiscasset, Me., on November 28, 1894, and is survived by his widow and three children, Myron H., Anne Louise (Mrs. Gordon V. Cox), and Mary Hallett (Mrs. Harry F. Jensen, Jr.); and by a sister, Mrs. Lena Barber. A brother, Joseph, died in 1938.

Mr. Atkinson was elected a Member of the American Society of Civil Engineers on April 4, 1911.

WILFRED KEEFER BARNARD, M. Am. Soc. C. E.[1]

DIED JANUARY 22, 1936

Wilfred Keefer Barnard was born in St. Joseph, Mo., on January 17, 1879. He was the son of the late John Fiske Barnard,[2] M. Am. Soc. C. E., and Julia Boswell (Keefer) Barnard, and came of engineering stock on both sides of his family. His father, a graduate of Rensselaer Polytechnic Institute, at Troy, N. Y., successfully followed railroad engineering to become the president and general manager of the Ohio and Mississippi Railroad (later the Baltimore and Ohio Southwestern Railroad Company). On his mother's side, he was a great nephew of the late Thomas C. Keefer, Past-President and Hon. M. Am. Soc. C. E., a noted Canadian engineer.

Mr. Barnard's early education was obtained in the public schools of St. Joseph, and of Cincinnati, Ohio. His engineering education was obtained at Sheffield Scientific School of Yale University, New Haven, Conn., from which he was graduated in June, 1901, with the degree of Bachelor of Philosophy.

Entering the service of the Pennsylvania Railroad Company the following month as engineering assistant, he was detailed to the Pittsburgh Division of the Pennsylvania Lines West of Pittsburgh, on track work and surveys. There he had charge of eight miles of second track, until October, 1903. He then became assistant superintendent of the Shirley Radiator and Foundry Company of Indianapolis, Ind., where he remained until January, 1904.

Hearing the call of the West, and with a returning desire to do railroad engineering, he entered the service of the San Pedro, Los Angeles and Salt

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[1] Memoir prepared by Charles T. Leeds, M. Am. Soc. C. E.

[2] For memoir, see *Transactions*, Am. Soc. C. E., Vol. LXVIII (December, 1910), p. 479.

Lake Railroad (later absorbed by the Union Pacific Railroad Company) where he remained until November, 1909. From February, 1904, to September, 1905, he served as assistant engineer (under the division engineer, Maintenance of Way) in charge of certain surveys and main line reconstruction on the Salt Lake Division, location, and construction of a seven-mile extension of the Frisco Branch, and Division Terminal construction, laying out and building the Caliente (Nev.) Yard, including the roundhouse, coaling trestle, water tank, and various buildings. In September, 1905, he was made assistant engineer, Maintenance of Way, of the Los Angeles Division, and was placed directly in charge of the engineer corps, drafting rooms, etc., and as assistant and acting engineer, Maintenance of Way, was in charge of general maintenance work, construction of branch lines, wharves, harbor surveys, bridges, buildings, signals, etc. It was during this period that the groundwork was laid for the present important marine terminal development of the Union Pacific Railroad Company at Los Angeles-Long Beach Harbor.

In November, 1909, he transferred his allegiance to the Pacific Electric Railway, with headquarters in Los Angeles, Calif., being made engineer of maintenance of way and structures, and was placed in charge (under the general manager) of 650 miles of urban and interurban track, bridges, buildings, etc., reconstruction (without interrupting traffic) of all bridges on the Mount Lowe Railway and of the terminal building at Los Angeles.

In March, 1913, he resigned from the railroad service to engage in private practice as a consulting engineer, in partnership with the writer, with offices in Los Angeles. This partnership continued until May, 1930.

Among the notable items of work handled by the firm of Leeds and Barnard during this period were:

Report on flood control of the Santa Ana River, California.

Report on and construction of flood control system for Coachella Valley, California.

Development of Newport Harbor, California.

Reconnaissance survey and land appraisal for a railroad from Gallup, N. Mex., into the San Juan Basin (Colorado, Utah, Arizona, and New Mexico).

Comprehensive plans for future development of Los Angeles-Long Beach Harbor ("Greater Harbor Committee of 200," Los Angeles Chamber of Commerce).

Report on shore protection for Venice, Calif.

Preliminary plans for Union Terminal Station, Los Angeles.

Construction of water supply system for Montecito County (Calif.) Water District, including driving Doulton Tunnel and constructing Buell Dam (hydraulic fill).

In May, 1930, the firm of Leeds and Barnard was consolidated with the firm of Quinton, Code and Hill, Consulting Engineers (William H. Code, Raymond A. Hill, and the late John H. Quinton, Members, Am. Soc. C. E., and the late Louis C. Hill, Past-President, Am. Soc. C. E.) under the name of

Quinton, Code and Hill-Leeds and Barnard. In this organization he continued until the time of his death.

On March 14, 1912, he was married to Katherine Tritle Clark, who, with their daughter, Katherine Elizabeth, and their son, John Fiske, survives him.

He always took a most active interest in Society affairs. He was a charter member of the Los Angeles Section, serving as its secretary from its formation in 1914 to 1916. His energy and public spirit contributed largely to the success of the Section in that formative period. He was elected vice-president of the Section in 1919, and its president in 1920.

He was also a member of the American Railway Engineering Association, the Society of Terminal Engineers, the American Water Works Association, the American Association of Port Authorities, and the American Concrete Institute, as well as the Beach Club of Santa Monica, Calif., the Flintridge Riding Club of Pasadena, Calif., and the California Club and the University Club of Los Angeles, serving as President of the latter in 1921–1922.

On his death, the Board of Directors of the Los Angeles Section of the Society, in extending their condolence to his wife, adopted the following resolution as expressing their feeling regarding him:

> "His professional life has been full of constant usefulness and unfailing helpfulness to the American Society of Civil Engineers in particular and to the Engineering Profession as a whole.
>
> "The immeasurable loss to the Los Angeles Section is not only due to the deprivation of his faithful and efficient services rendered as its Secretary, Vice-President and President, but also for his genial personality and whole hearted cooperation in all of its activities."

There appears never to have been a question as to his choice of a profession, though very wisely he took a preliminary "taste" by serving as rodman with a survey crew on the Burlington Railroad during the summer before entering college. Through his father and his great uncle, he was fully appreciative of the significance of the engineering profession, its opportunities, its difficulties, and its obligations. This appreciation continued throughout his life.

Always public-spirited, he gave of his best efforts to far-seeing improvements. To him is due a major part of the credit for the unification of the railroads serving Los Angeles-Long Beach Harbor. The writer knows, from personal association with him in that project, the skill and diplomacy which were necessary to enable him to coordinate the discordant views. Patiently overcoming the initial jealous opposition of representatives of discordant interests, he succeeded in achieving a spirit of cooperation which eventually resulted in the formation of the present Harbor Belt Railroad now efficiently serving the port, and the adoption of a comprehensive plan of rail, water, and highway service for future development in this fast-growing region.

Similarly he was one of the early proponents of a Union Terminal to serve all the railroads entering Los Angeles. Unfortunately, he did not live to see the fruition of his efforts. The plan finally adopted and ordered by the State Railroad Commission, however, was essentially that first proposed by him.

He was a man who loved his home and his family. He also felt keenly his obligation to his fellow men. His career was made the means of enlarging and enriching all his human relations. This was evidenced not only by his wide circle of friends, but by his untiring efforts as Secretary of the Los Angeles Section of the Society during its early and formative years, from which he rose soon to be its President. He also was selected as president of the University Club of Los Angeles when, in the face of a major financial depression, that organization resolutely determined to construct new quarters and a leader was needed to guide its affairs through that difficult period.

His health was not good at the time, and his conscientious, untiring efforts in reconciling disputes between architect and contractor, with inevitable criticism from Club members in a period of economic depression, are believed by many to have contributed to the physical breakdown which later caused his death.

Mr. Barnard was a man of great force of character and the utmost honesty and integrity. He more than once remarked: "Not every engineer can achieve a reputation for brilliancy, but he can become noted for being always square." He had no patience with dishonesty or graft, no matter how trifling, and woe betide the contractor who tried to cover up dishonest work or to evade the strict provisions of the specifications. On the other hand, he was charitable with failings of others and would not judge a man without first hearing and making allowance for all the facts. He left to his profession, and to all who knew him, that finest contribution that any man can make—an example of moral integrity, generosity, and public spirit.

He was a man of far deeper religious convictions than many realized. For though all knew him to hold the highest standards of personal and public integrity, he never boasted. No one ever questioned where he stood on a matter of principle.

The period of illness which preceded his death afforded him the opportunity to write, solely for his own satisfaction, certain thoughts which he had not expressed publicly, but which show the true nature of the man. These were not discovered until after his death, but they now leave a revealing picture of his real character, and the actuating principles that guided his life. The first shows his deep and earnest faith:

THE FOOL

Sayeth the fool in his heart, "There is no God";
Poor witless thing! enclothed in motley, cap and bells,
The tinkling symbols of his earthly gain;
He gives pronouncement to these words,
The futile product of his finite brain.
Poor Fool!

The rich man's fields produced such grain
That it filled his barns and he built again.
Wine from his vineyards brimmed full on the lees,
"Eat, drink and be merry and take thine ease,"
Said he. Said the Lord, "This night shall be
Thy greedy soul required of thee."
Poor Fool.

But the penitent thief on the other cross
His soul burned clean of earthly dross
Sees God in the Man beside him there,
And from his soul he breathes a prayer:
"Remember me"—
And God's voice through His Son on the Cross replies:
"This day shalt thou be in Paradise with me."

W. K. B.

With this as a foundation it is not difficult to understand the faith and courage of the man who, as he neared the end of his career, wrote the following:

CARRY ON

As men give way to doubt and fear,
As men give way to doubt and fear,
God keep within our souls alive
That spark, which gives us power to strive
And carry on!

Our fathers fought for liberty,
The right of every man to rise.
Let's not in haste cast all aside
But by experience abide
And carry on!

Help us, O God, to save our souls
And through these days of change and stress
To take the way that spares us less,
To keep from every littleness
And carry on!

W. K. B.

Mr. Barnard was elected an Associate Member of the American Society of Civil Engineers on November 8, 1909, and a Member on July 2, 1913.

JAMES GARNETT BASINGER, M. Am. Soc. C. E.[1]

DIED MARCH 13, 1941

James Garnett Basinger was born on May 27, 1870, in Savannah, Ga., the son of William Starr and Margaret (Garnett) Basinger. He was graduated from the University of Georgia, at Athens, Ga., and received his degree of Master of Arts in 1889, followed by the professional degree of Civil and Mining Engineer in 1900.

After a short experience in general engineering work, he joined the staff of the Topographical Bureau of the 23d and 24th wards of the City of New York, N. Y., an experience that has been shared and valued by many of New York's engineers. In 1896 he entered the Engineering Department of the Department of Docks and Ferries, in the City of New York, and from that time until his retirement he was closely identified with waterfront engineering, principally in the vicinity of New York City. While with the city he was in charge of the North River waterfront from 1901 to 1905, and during these years he constructed many sections of granite-faced quay wall of designs such as solid concrete walls, resting on rock at depths of water up to 30 ft; thin walls supported on piles cut off 15 ft below low water with relieving platforms; and combinations and variations of these and other designs. During this time he also built many piers for steamship terminals, some of them on piles more than 100 ft in length, some with wooden and some with concrete decks, and pier sheds both of wood and of steel.

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[1] Memoir prepared by Charles E. Trout, M. Am. Soc. C. E.

As an engineer for the city, for various contractors, and for waterfront owners, he was for nearly forty years a well-known figure in harbor work on the North Atlantic Coast. His work included design, construction, valuation testimony in court, and engineering work of all kinds on projects large and small connected with the development and use of the waterfront.

In 1905 and 1906 he was in charge, for George B. Spearin, of the construction of Dry Dock 4 at the New York Navy Yard, where he devised and used a new type of steel sheet piling, one of the earlier uses of this material.

Mr. Basinger entered private practice as a consulting engineer in his chosen field in 1906. In this capacity he continued his work in harbor and waterfront design and construction, although he did not confine himself strictly to such activities, some of his outstanding work being inland, such as boulevard design, power plant investigation, etc.

During the first World War, he was in charge of the supervision, for the construction quartermaster, of all construction at the Army Supply Base in South Brooklyn, N. Y. This development, costing some thirty million dollars, and consisting of two very large concrete warehouses, four steamship piers more than 1,200 ft long with two-story steel sheds on three of them, railroad yards, paved streets, water supply, sewerage, lighting, and dredging, was finished in about fifteen months. After completion of Mr. Basinger's service on this project, he returned to private practice.

His activities extended from the islands of St. Pierre and Miquelon, off the south coast of Newfoundland, where he was in charge of the construction of a coaling pier, to Brunswick, Ga., where he was connected with the design and construction of a monumental boulevard. His experience covered a great variety of work at many places between these points. His principal activity, however, was in and about the City of New York where, through the years, he was engaged on docks, transit sheds, breakwaters, shore protection work, the filling of low lands, development of waste lands, dredging, paving, railroad work, and everything having to do with waterfront development for either public or private use. His clients included most of the railroads entering New York, many steamship companies and industrial corporations, the United States, the City of New York, other engineers, architects, contractors, many private waterfront owners, and others.

Of all his work, perhaps the project in which Mr. Basinger was more interested than any other was the construction of the new terminal for the Ocean

Steamship Company in Savannah in 1915 and 1916. He acted as designing and supervising engineer on this work, which represented an expenditure of approximately $1,000,000. It included a slip, 225 ft wide by 1,020 ft long, with a depth of water at mean low tide of 26 ft; a reinforced-concrete bulkhead with a total length of 2,590 ft; steel freight sheds; an office building; and a system of tracks under the sheds that had a total length of two miles.[2]

In addition to his membership in the Society, Mr. Basinger was a member of the American Society of Mechanical Engineers, Merchants Association, New York Athletic Club, Sigma Alpha Epsilon, and Phi Beta Kappa.

He was married on April 25, 1905, to Nannie Lloyd Screven of Savannah, who, with one daughter, Anne Lloyd, survives him.

Mr. Basinger was elected an Associate Member of the American Society of Civil Engineers on October 3, 1900, and a Member on April 30, 1907.

CLAUDE HALE BIRDSEYE, M. Am. Soc. C. E.[1]

DIED MAY 30, 1941

Claude Hale Birdseye, former chief topographic engineer of the United States Geological Survey, distinguished soldier in the First World War, eminent photogrammetrist, and leader of a daring exploration through the Grand Canyon of the Colorado River, leaves behind him a life crowned with high achievements.

He was born in Syracuse, N. Y., on February 13, 1878, and was the son of Katherine Lamb (Hale) and George Frederick Hurd Birdseye. The family moved to Oberlin, Ohio, and Claude was graduated from Oberlin High School in June, 1895. He attended Oberlin Academy the following year, and in 1901 received a Bachelor of Arts degree from Oberlin College. He was an instructor in physics at the University of Cincinnati, Cincinnati, Ohio, in the spring of 1901, and later in that year he worked as an assistant in a topographic field party of the United States Geological Survey. Then in the fall he enrolled at Ohio State University, Columbus, Ohio, for postgraduate study in engineering, and in July, 1902, he again joined a field party of the Survey.

Determined to pursue his interest in mathematics and the natural sciences he accepted, on October 19, 1904, an appointment as topographic aide, classified in the Federal Civil Service, for permanent employment in the Geological Survey. The salary was small, but his optimism was great. Only a month after his appointment he returned to Oberlin and was married to Grace Gardner Whitney on November 21, 1904. Thus Mr. Birdseye began, almost simultaneously, his domestic and professional careers which together brought him through life to high success.

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[2] "Design and Construction Features of the Ocean Steamship Co.'s Terminal at Savannah, Ga.," *Engineering and Contracting,* November 3, 1915, p. 343.

[1] Memoir prepared by R. M. Wilson, M. Am. Soc. C. E.

Mr. Birdseye's experience and skill in mapping the geographical features of his country were gained by the hard work of actual practice in the mountains and forests. His early assignments in the Geological Survey were not the easy ones, though many of them were routine. His natural resourcefulness and ability soon became so apparent, however, that he was more and more frequently chosen for special assignments of a kind that gave scope to his venturesome spirit. He was put in charge of the topographic mapping in Hawaii from 1909 to 1912, a task which involved the mapping of the active volcano, Kilauea, with its lake of fire. In the following year (1913) he supervised the mapping of Mount Rainier, Washington. On the summit of that mountain, 14,408 ft above sea level, his party was caught in a blizzard and was forced to seek precarious refuge in a steam-vent.

In January, 1916, Mr. Birdseye was appointed geographer in charge of all topographic mapping by the Geological Survey in its Rocky Mountain Division.

The readiness to cope with emergencies, and the power to command, that were gained in such assignments as these laid the foundation for Mr. Birdseye's valuable service to his country in the First World War. He was called into active military service on June 18, 1917, as a captain of engineers, and was promoted about a month later to the rank of major. He sailed for France on August 16, 1917, for duty as orienteur officer with the First Brigade of Coast Artillery, applying his knowledge to the surveys and observations necessary for aiming the big guns. A year later he was promoted to the rank of lieutenant-colonel and assigned to the Office of the Chief of Coast Artillery, American Expeditionary Forces, in charge of artillery ranging and map information.

These rapid promotions, and the fact that instruction manuals[2] written by him were published by the War Department, show that his abilities were productive and appreciated. His achievements were further acknowledged in 1919 when the French Order of University Palms, grade of Officier de l'Instruction Publique, was conferred upon him.

After the War he obtained his discharge, but retained a reserve officer's commission as lieutenant-colonel. He returned to the Geological Survey, and on October 1, 1919, became chief topographic engineer, thus continuing the line of endeavor begun in his college days.

In this new and responsible position he directed all field and office work of making the topographic "quadrangle" maps that are familiar to engineers throughout the United States. The art of mapping was entering a critical period, for the use of airplanes and aerial photography, which had developed so rapidly during the War, was beginning to alter the technique of mapping through the introduction of photogrammetric methods. Colonel Birdseye's adaptability, experience, and initiative qualified him to understand the value of these methods and to guide the Geological Survey into its modern mapping practices. His studies and his administrative direction in the uses of aerial photography soon made him one of the foremost photogrammetrists in the United States. His character counted for as much as his ability—he was so unfailingly considerate and understanding that he won the wholehearted loyalty of his associates and inspired in them a high degree of cooperation. It is fortunate indeed that so able a leader was in charge of the Topographic Branch during that period of transition.

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[2] "Meridian Determination," Pt. III of Orientation for Coast Artillery, Coast Artillery Training Center, Fort Monroe, Va., 1919; also Appendix I to Pt. III, "Ephemeris of the Sun and Stars," and Appendix II to Pt. III, "Circumpolar Ephemeris."

In 1923 Colonel Birdseye's untamed spirit of adventure again broke forth, and he took time from his administrative duties to lead an expedition by boat down the rough waters of the Colorado River through the Grand Canyon.[3],[4] The reconnaissance surveys made by the expedition were highly useful in choosing the site for Hoover Dam. For this and other contributions to American geography he was awarded the Charles P. Daly medal by the American Geographical Society on April 22, 1924.[5]

Continuing as chief topographic engineer, he found time during his efficient administration of the Topographic Branch to compile, revise, and enlarge the "Topographic Instructions," published in 1928 as *Geological Survey Bulletin No. 788,* and prepared "Formulas and Tables for the Construction of Polyconic Projections," which were published as *Geological Survey Bulletin No. 809* in 1929. These are the standard manuals for topographic mapping. Colonel Birdseye also wrote a number of shorter articles which were published in various technical periodicals; as, for example, "Surveying and Mapping in the United States."[6]

He worked tirelessly and wrote assiduously in connection with his society and committee memberships; and he preferred to direct his energy toward the solving of new problems rather than the describing of old ones. These activities gave him a wide range of valuable contacts with other engineers, geographers, and photogrammetrists.

In September, 1929, he resigned from the Geological Survey to become president of the Aerotopograph Corporation of America, and directed the detailed photogrammetric surveys of the site for Hoover Dam. He described this work, and other general features of photogrammetry, in papers prepared or published about this time.[7], [8]

His achievements in the thirty years after graduation were recognized by Oberlin College in June, 1931, when the honorary degree of Doctor of Science was conferred upon him with these words, spoken by President Ernest H. Wilkins: "Claude Hale Birdseye, traverser of uncharted ways of river and of air, recorder of the eagle's vision for the service of mankind * * *"

Colonel Birdseye returned to the Geological Survey on February 1, 1932, as engineering assistant to the director, and a few months later became the chief of the Division of Engraving and Printing, where he was in charge of the reproduction of the maps published by the Survey. Meanwhile his interest in the larger fields of surveying, mapping, and photogrammetry continued, and he worked upon various important general problems.[9] He was chairman of the committee that prepared the fundamental and detailed "National Mapping Plan" proposed by the Federal Board of Surveys and Maps in November, 1934.[10] His committee also prepared a further report on the same subject which was adopted by the Board on December 8, 1936. These reports were reviewed by the Secretary of the Interior in a letter addressed to Congress on January 28, 1937,[11] in which he urged that the mapping of the United States be accelerated.

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[3] "A Boat Voyage through the Grand Canyon of the Colorado," by Claude H. Birdseye, and Raymond C. Moore, *The Geographical Review,* Vol. XIV, No. 2, April, 1924.

[4] "Surveying the Grand Canyon of the Colorado," by Lewis R. Freeman, *National Geographic Magazine,* May, 1924.

[5] *The Geographical Review,* Vol. XIV, No. 3, July, 1924, p. 465.

[6] *Transactions,* Am. Soc. C. E., Vol. 92 (1928), p. 1486.

[7] "Photographic Surveys of Hoover Dam Site," by C. H. Birdseye, *Civil Engineering,* Vol. 1, No. 7, April, 1931, p. 619.

[8] "Aerial Stereo-Photographic Mapping Instruments and Methods," by C. H. Birdseye, *Journal of the Optical Society of America,* March, 1932.

From 1930 to 1935 Colonel Birdseye was particularly active in delivering lectures before engineers' clubs and groups of engineering students at colleges and universities. Some of his subjects were: "Modern Methods in Aerial Photography," "Aerial Photographic Mapping," "New Developments in Topographic Mapping," "Photographic Surveying and Mapping," and "Hoover Dam-Site Surveys." An article published in 1935[12] also deals with similar subjects.

He was selected as one of the technical advisers who prepared the "Standard Specifications for Aerial Photography for General Map Work and Land Studies" for the Procurement Division of the Treasury Department. These specifications were adopted officially in 1937, and brought order out of chaos in drafting contracts for the vast amount of aerial photography used by different government agencies for a wide variety of purposes.

On April 30, 1938, he was commissioned colonel in the Engineer Corps, U. S. Army Reserve, advancing from the rank of lieutenant-colonel held since his discharge from active military service. The promotion was initiated by the War Department in recognition of his increasing prominence in a field of engineering that is so important in time of war.

Colonel Birdseye went to Europe in June, 1938, to study the methods used there for engraving and reproducing maps. He attended the International Geographical Congress at Amsterdam, the Netherlands, and had hoped to attend the Congress of the International Society of Photogrammetry at Rome, Italy, but after visiting Southampton, England, and Jena, Thuringia, he was forced by ill health to curtail his trip. Nevertheless, he brought back information that will be of lasting value to the science of geography and the art of mapping in the United States.

Colonel Birdseye was a valued and active member in the following organizations:

Washington Society of Engineers (President, 1938; Director, 1920–1921, 1929–1930, 1939–1940)

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[9] "Stereo-Topographic Mapping," by C. H. Birdseye, *Transactions*, Am. Soc. C. E., Vol. 98 (1933), p. 771.

[10] "The National Mapping Plan," *Civil Engineering*, Vol. 5, No. 2, February, 1935, p. 125.

[11] "Topographic Mapping of the United States," S. Doc. No. 14, 75th Cong., 1st Session.

[12] "Plotting Maps from Aerial Photographs," by C. H. Birdseye, *Engineering and Mining Journal*, Vol. 136, No. 11, November, 1935.

Washington Academy of Sciences
National Research Council, Division of Geology and Geography
American Geophysical Union
Society of American Military Engineers (Treasurer, 1929 and 1935)
American Geographical Society (Fellow)
Association of American Geographers (President, 1939)
American Society of Photogrammetry (a founder; first President, 1933–1934)
American Engineering Council
Federal Board of Surveys and Maps (Secretary, 1920–1924; Chairman, 1926)
Canadian Institute of Surveying
International Geographical Union (Vice President, 1938)
National Geographic Society
Royal Geographical Society (Fellow)
International Society of Photogrammetry
American Legion (Maryland, Post No. 105)
Cosmos Club, Washington, D. C.
Explorers' Club, New York, N. Y.
Kappa Sigma Fraternity

His presidential address[13] delivered before the Association of American Geographers at Chicago in 1939 was of outstanding interest, being a comprehensive review of the methods used in mapping by photogrammetry.

Although the foregoing is but an incomplete recital of Colonel Birdseye's many and valuable contributions to engineering, geography, and photogrammetry, it shows how steadfastly and how effectively his untiring energy and ability were directed toward high scientific objectives.

This great man, however, must be credited with other achievements of an even higher order than those he gave to science directly, achievements of a kind that will perpetuate his memory in the hearts of all who knew him. By friendly counsel and assistance he was able to inspire many with enthusiasm for greater attainments; thus did his spirit generously pervade the lives of his associates. His own career is ended now, but its powerful effect will long continue in the enriched careers of others. Such were the human qualities and achievements that made him a true leader of men; such were the features of his character that will live with us always as cherished memories.

Colonel Birdseye died at Mt. Alto Veterans' Hospital in Washington, D. C., on May 30, 1941. He is survived by his widow, Grace Gardner Whitney Birdseye, his three children, Charles, Frederick, and Florence Birdseye, and his brother, Sidney Birdseye.

Colonel Birdseye was elected an Associate Member of the American Society of Civil Engineers on April 30, 1912, and a Member on October 15, 1923.

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[13] "Stereoscopic Photographic Mapping," by C. H. Birdseye, *Annals of the Association of American Geographers*, Vol. XXX, No. 1, March, 1940.

JOHN BENJAMIN BODY, M. Am. Soc. C. E.[1]

Died May 23, 1940

John Benjamin Body was born on February 27, 1867, at Mevagissey, Cornwall, England. He was the son of John and Mary Ann (Harris) Body. He was educated at City of London College, London, under Professor Henry Adams.

After six years of training in Great Britain, Mr. Body proceeded to Mexico. His choice of a country in the Western Hemisphere for his work in the profession was the outstanding feature of his career. From the date of that choice until his death he devoted his professional services to the interests of one firm only. Thus, from 1890 to 1940 he identified himself with the activities of Messrs. S. Pearson and Son, Ltd., Civil Engineering Contractors. This is the dominant factor of his life, and a recital of the great works executed by that firm in Mexico is necessarily a statement of the achievements of J. B. Body in the profession, because under the direction of the late Weetman D. Pearson, the first Viscount Cowdray, all of these works were brought to fruition. The following are the chief activities in which Mr. Body participated:

The drainage of the Valley of Mexico (1890 to 1895): This was accomplished by a canal 30 miles long, 20 ft to 70 ft deep, costing $7,500,000.

Harbor Works at Veracruz, Mexico: This required 260,000 cu yd of masonry, 1,000,000 tons of rock in breakwaters, and 12,000,000 cu yd of dredging in the inner basin. Subsidiary works for the city's sanitation and water supply and for the port, dock, and warehouse equipment were involved. The total value of the works was $12,500,000. These works were in progress under Mr. Body from 1896 to 1899, and involved a serious struggle against yellow fever and malaria.

Trans-isthmian railway at Tehuantepec, Mexico: This standard gage railway was constructed together with the harbors of Coatzacoalcos on the Gulf of Mexico and Salina Cruz on the Pacific Ocean, a distance of 210 miles. As a large part of it was through swamps, its construction presented problems. The two ports comprised extensive harbor and dock works. On the Gulf side, 500,000 cu yd of rock were placed in breakwaters, and 3,300 ft of deep-water, screw-pile, steel wharves were built that were fully equipped with warehouses, electric wharf cranes, and similar features. On the Pacific side a heavy breakwater 3,300 ft long, composed of rock and concrete blocks laid in sloping bond in 55 ft of water, was built. An inner basin, 3,300 ft by 750 ft, and a dry dock, 590 ft by 72 ft with 29.5 ft over the sill, were included in the port works. The principal wall of the basin, 3,300 ft long, was fully equipped with warehouses and electric wharf cranes. The fight against yellow fever and malaria continued during the progress of these works, and the former fell disease was eventually stamped out and the incidence of malaria much reduced. The railway and ports were of modern character with ample capacity to handle the heavy trans-isthmian traffic of goods between Eastern and Western United States and the Hawaiian Islands before the completion of the Panama Canal and the diversion of the traffic to it in 1914. Mr. Body was engaged on these works from 1899 to 1901, and they cost $15,000,000.

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[1] Memoir prepared by T. L. Bonstow, M. Am. Soc. C. E.

Development of petroleum production in Mexico: The establishment of the Mexican Eagle Oil Company next occupied the attention of Messrs. S. Pearson and Son and Mr. Body. The necessity of a local, cheap, fuel-oil supply for the trans-isthmian railway locomotives and the electricity generating plants of the ports was the incentive for this development. Petroleum was soon found on the Isthmus of Tehuantepec but not in sufficient quantities at that time for the purpose in view. The search was widened and immense quantities of petroleum were found in Northeastern Mexico, and, at a much later date, on the Isthmus itself, which necessitated the construction of pipe lines, pump stations, refineries, storage tanks, oil loading wharves, and ocean-going tankers, in addition to inland and water transportation. In the course of time the products of the Mexican Eagle Oil Company found their way into every corner of the globe. Mr. Body was intimately concerned with all the development work in this great undertaking. A maximum production of 20,000 tons of petroleum daily was reached, all of which was turned into fuel oil, asphalt, lubricating oil, gas oil, kerosene, gasoline, paraffin wax, and other industrial products. All of the refining was done in Mexico with the manufacture of the packages required. Distribution was organized within Mexico and to all the principal consumption centers of the world. His interest in this remarkable development continued until its activities within the republic were taken over by the Government of Mexico in 1938.

For twenty-five years Mr. Body resided in Mexico. Following his first crossing of the Atlantic in 1890 he continuously cultivated the closest relations of friendship and business with colleagues and friends in the United States, in which country he also had interests in the American Petroleum Industry.

In temperament Mr. Body was democratic and genial. He had the gift of always getting the best out of his associates and assistants. In 1915 he returned to England where he resided until his death. However, during the twenty-five years, 1915 to 1940, he made many trips to the United States and Mexico.

On August 10, 1898, he was married to Mary Hamer, who, with one daughter, Beatrice Millicent Kelton, survives him.

Mr. Body was elected an Associate Member of the American Society of Civil Engineers on October 2, 1895, and a Member on May 2, 1900.

WILLIAM BOWIE, M. Am. Soc. C. E.[1]

DIED AUGUST 28, 1940

William Bowie was born in Anne Arundel County, Maryland, on May 6, 1872, the son of Thomas John and Susanna (Anderson) Bowie. He was a descendant of an old Colonial family that contributed much to the development and history of Maryland. Born on a farm, he received his early education in the district school and at St. John's College, in Annapolis, Md. He was graduated from Trinity College, in Hartford, Conn., in 1893 with the degree of Bachelor of Science and from Lehigh University, at Bethlehem, Pa., in 1895 with the degree of Civil Engineer. He later received honorary degrees from Trinity College (Master of Science in 1907 and Doctor of Science in 1919); Lehigh University (Doctor of Science in 1922); University of Edinburgh, in Edinburgh, Scotland (Doctor of Laws in 1936); and George Washington University, in Washington, D. C. (Doctor of Laws in 1937).

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[1] Memoir prepared by Henry G. Avers, Assoc. M. Am. Soc. C. E.

Practically all of Mr. Bowie's professional life was spent in the service of the U. S. Coast and Geodetic Survey, which he entered on July 1, 1895. He was engaged on topographic surveys in Buzzards Bay, Mass., in northern Chesapeake Bay, and in San Juan Harbor, Puerto Rico, until 1899. From January to May, 1900, he was engaged on extending a scheme of triangulation across Puerto Rico from Ponce to San Juan. This triangulation has assumed considerable importance because the distance across the island determined by triangulation was found to be one mile shorter than that computed from the latitude observations at the two ends of the scheme, thus showing to a remarkable degree the effect of the deflection of the vertical upon astronomic observations.

From July, 1900, he was engaged successively in the measurement of base lines along the 98th meridian; general surveys in Prince William Sound, Alaska; and triangulation along the 98th meridian in Kansas and Texas. In January, 1903, he began a tour of duty in the Philippine Islands, where he took part in general surveys in Lingayen Gulf, Batangas Bay, Manila Bay, Balayan Bay, and in the vicinity of Iloilo.

Returning to the United States in June, 1905, he again took up geodetic work and was engaged, in order, on the determination of longitudes in Florida, Georgia, and South Carolina; triangulation in Minnesota; and observations for latitude and azimuth in Indiana, Michigan, Illinois, Wisconsin, and Oklahoma. He concluded his field assignments with observations on the eastern end of the Texas-California arc of primary triangulation, which was completed in January, 1909.

Conditions in the field during the period of Mr. Bowie's service were far different from what they are now (1941). His were really the "horse-and-buggy" days, and travel between triangulation stations, often 50 miles apart, was solely by horse-drawn vehicles. Few were the comforts that a party working under these conditions could enjoy, but his habitual cheerfulness carried him through many difficulties, and his surveys were made with a thoroughness, accuracy, and economy of execution that characterized all the undertakings of his career.

Returning to Washington, D. C., Mr. Bowie was appointed assistant to the late John F. Hayford,[2] M. Am. Soc. C. E., and succeeded him as chief of the Computing Division and inspector of geodetic work in October, 1909. A few years later, these two offices were merged in that of chief, Division of Geodesy, a position which he held until his retirement on December 31, 1936, and interrupted only by his service as Major in the Corps of Engineers, U. S. Army, from August, 1918, to February, 1919.

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[2] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1614.

Following his retirement, he was appointed executive secretary of the Society of American Military Engineers in December, 1939, and served in that capacity, and as editor of the Society's *Journal,* until his death.

At the time when Mr. Bowie became associated with Mr. Hayford, the latter was completing his supplementary investigation of the figure of the earth and isostasy. Entering into the study of isostasy and related subjects with his usual vigorous interest, Mr. Bowie collaborated with Mr. Hayford in the publication of the results of an investigation of the effect of topography and isostatic compensation on the intensity of gravity.[3] Thus began the studies in subjects to which he was to devote much of his time in the years to come and which were to result in his becoming a champion of the theory of isostasy. As an advocate of this theory, he conducted, through correspondence, lectures before scientific organizations, and conferences, a campaign that brought about the acceptance of his views, in principle at least, by many of the leading geodesists and geologists of the world.

His varied experiences in the field work of the U. S. Coast and Geodetic Survey eminently fitted him for his position as chief of the Division of Geodesy. Under his progressive leadership and inspiring guidance, methods and equipment were modernized and marked advances were made in both field and office procedure. Among the notable achievements during his tenure of office may be mentioned the adoption of the U. S. Standard Datum in 1913 by Canada and Mexico. This is the only instance where the triangulation of so large a part of a continent is referred to a single datum. Owing to the international character of the datum its name was changed to that of North American Datum. In 1928, through the cooperation of Canada, this datum was extended to Alaska.

A large step forward was made in 1924 when it was decided to readjust the triangulation net of the country. As arcs were added to the net, they had been adjusted to fit the existing triangulation, with the result that, as the net expanded and loops were formed, the last arc to be fitted in received corrections much larger than should be ordinarily applied to this class of work. To distribute the corrections more evenly, it would have been necessary to adjust the entire network of triangulation as a whole, but to do this by the methods then prevailing would have required more time and work than could have been devoted to it. Confronted with this problem, Major Bowie suggested a method, later known as the Bowie Method of Triangulation Adjustment, in which junction figures are used, and the adjustment divided into a number of different parts that can be made simultaneously. This made it possible to complete a large adjustment within a reasonable length of time. The method was applied first to all the triangulation west of the 98th meridian of longitude, consisting of 16,000 lin miles of arcs, and later applied to the triangulation east of that meridian, having 12,000 lin miles of arcs. This framework of triangulation as thus adjusted is the basic control for all geographic positions in the United States, and it has proved to be so accurately observed and adjusted that new arcs are fitted to it without undue distortion.

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[3] "The Effect of Topography and Isostatic Compensation Upon the Intensity of Gravity," *Special Publication No. 10,* U. S. Coast and Geodetic Survey, Washington, D. C., 1910.

The adoption in 1927 of the Bilby steel tower for elevating the observer and instrument above obstructions in the line of sight made possible the continuance of triangulation operations, regardless of the character of the terrain. Before the advent of the steel tower, the extension of triangulation over flat, wooded country through the use of wooden towers was becoming so costly that the substitution of traverse was seriously considered. The wooden towers erected at the triangulation stations could be used but once and their building required several days. The steel towers can be erected in a few hours, taken down, and used repeatedly at other stations. This is one of the many instances where the encouragement given his subordinates by Major Bowie resulted in improvements in geodetic methods and instruments.

Major Bowie was constantly alert to the possibilities of extending the utility of geodetic work. With this in mind he inaugurated the establishment of azimuth marks at triangulation stations, as starting points for local surveys; the publication of distances and elevations in feet and meters; the stamping of elevations on bench marks; and the computation of plane, in addition to spherical, coordinates for triangulation stations. The introduction of plane coordinates met immediately with a popular response from surveyors to whom spherical coordinates with their attendant complicated computations had little or no appeal. Plane coordinate systems were devised for each state in the United States, and the simplicity of their use as a reference for property boundary lines and local surveys has resulted in their legal adoption in six of the states.

In connection with his work in the Division of Geodesy, Major Bowie became interested in the mapping of the United States and early recognized the woeful lack of adequate maps showing general topography and natural and artificial boundaries. Mapping probably had no more ardent crusader in the country, and with the assistance of technical societies and prominent statesmen, whose support he succeeded in enlisting, the great impetus given mapping in recent years can be said to be mainly attributable to his efforts.

Major Bowie was well known to geodesists abroad. He took part in the conference of the International Geodetic Association at Hamburg, Germany, in 1912, the last conference of that organization before the first World War.

After the war, plans were made to organize international science on a more comprehensive basis. For this purpose a conference was held at Brussels, Belgium, in 1919. Major Bowie was a prominent member of the American delegation and had an important part in shaping the decisions reached by the conference. As one result of the conference, an International Union of Geodesy and Geophysics was formed. Its seven sections cared for the various branches in geophysics and for geodesy. From the first conference in Rome, Italy, in 1922, and thereafter, Major Bowie had a leading part in the affairs of the Union and of the Section (later changed to Association) of Geodesy, which took the place of the prewar International Geodetic Association. He was President of the Section and Association of Geodesy from 1922 to 1933, attending the conferences held at Rome, Italy; Madrid, Spain; Prague, Czechoslovakia; and Stockholm, Sweden.

Pressure of work at home in connection with the increased activity of the Coast and Geodetic Survey under the Civil Works Administration program prevented him from attending the Lisbon (Portugal) conference in 1933, but at that conference he was elected President of the entire Union of Geodesy and Geophysics. This office he held until 1936, presiding over the conference at Edinburgh in that year.

For his many valuable contributions to the advancement of his profession, Major Bowie received many well-deserved honors. He was awarded the Elliott Cresson Medal in 1937 by the Franklin Institute of Philadelphia, Pa., for his contributions to the science of geodesy. He was also awarded the Charles Lagrange Prize by the Royal Academy of Belgium in 1932; made an officer in the Order of Orange-Nassau by the Queen of the Netherlands in 1937; and received the decoration of the Cross of Grand Officer of the Order of St. Sava from Yugoslavia in 1939. The first impression of the medal of the American Geophysical Union, known as the William Bowie Medal and established as an award for distinguished attainment and outstanding contribution to the advancement of cooperative research in fundamental geophysics, was presented to Major Bowie at the meeting of the Union in April, 1939.

A prolific writer, Major Bowie was the author of the book, *Isostasy;* twenty-three government publications; and nearly three hundred separate articles published in technical journals and newspapers.

He was interested in many scientific societies and organizations in which he took an active part. He was President of the Washington Society of Engineers in 1914; President of the Philosophical Society of Washington in 1926; President of the Washington Academy of Sciences in 1930; chairman of the American Geophysical Union from 1919 to 1922 and again from 1929 to 1932; chairman of the Board of Surveys and Maps of the federal government from 1922 to 1924; member of the Committee on Surveying and Mapping, American Engineering Council; President of the Society of American Military Engineers in 1938; chairman of the Division of Surveying and Mapping of the American Society of Civil Engineers since its organization in 1926; President of the District of Columbia Chapter of the Society of Sigma Xi from 1935 to 1936; Honorary President of the Pan American Institute of Geography and History from 1929 until his death; President of the International Association of Geodesy from 1919 to 1933; President of the International Union of Geodesy and Geophysics from 1933 to 1936; corresponding member of the Académie des Sciences, Institut de France; corresponding member of the Russian Geographical Society; and foreign member of the Norske Videnskaps Akademiet, in Oslo, Norway. He was a member of the Army and Navy Club and the Cosmos Club of Washington, D. C.

Major Bowie was a man of high principles and fine character. He was endowed with a keen intellect, broad vision, and pleasing personality and had the happy faculty of devoting his entire energy to the matter before him, whether at work or at his favorite pastimes of billiards and bridge. He was considerate of his subordinates and enjoyed the affection and respect of all who were associated with him. His cheery greetings and genial companionship will be missed by a host of friends.

On June 28, 1899, he was married to Elizabeth Taylor Wattles, who survives him. He is survived also by a son, Clagett Bowie; and two brothers, John Bowie and Major Edward Bowie.

Funeral services were held at St. Thomas Episcopal Church, in Washington, D. C., on August 30, 1940, followed by burial with military honors in Arlington National Cemetery.

Major Bowie was elected a Member of the American Society of Civil Engineers on March 13, 1917.

LEWIS VAN CARPENTER, M. Am. Soc. C. E.[1]

DIED MAY 10, 1940

Lewis Van Carpenter, the son of John and Ollie (Carpenter) Carpenter, was born in Wheeling, W. Va., on December 23, 1894. He spent most of his early years through young manhood in his native city, receiving his elementary and secondary education in its public schools, and for a time working in one of the local steel mills. This latter experience doubtless accounted in part for one of his outstanding characteristics in his later professional life—an ability to secure and hold the respect and loyalty of subordinates among the non-professional groups engaged in construction and other enterprises that he supervised.

His higher education was secured the "hard way," as he earned most of his support through college by working as assistant engineer and draftsman with state, county, and municipal road and sanitary authorities in West Virginia. He received the degree of Bachelor of Science in Civil Engineering from the University of West Virginia, at Morgantown, in 1918, the degree of Master of Science from the same institution in 1925, and the degree of Master of Science in Sanitary Engineering from the University of Illinois, at Urbana, in 1926.

One of the first to volunteer for service in the first World War, Lewis Carpenter went overseas with the 304th Engineers. He saw active service in France, and was wounded while in action in the Meuse-Argonne battle on September 26, 1918. He was honorably discharged from the army in 1919, and from then until December, 1920, he was engaged in road work and later in the design and construction of sewers for Thermopolis, Wyo. During most of 1921 and 1922 he was safety engineer for the Semet-Solvay Company.

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[1] Memoir prepared by Thorndike Saville, M. Am. Soc. C. E.

With this varied background of work and experience Mr. Carpenter returned to the University of West Virginia in 1922, was appointed to the instructing staff, and was promoted successively to assistant professor, professor, and head of the Department of Civil Engineering. He occupied the latter positions until 1935, specializing in sanitary engineering. During his association with the University of West Virginia Professor Carpenter was the author of some twenty-seven articles in engineering publications, contributing particularly to knowledge concerning manganese in water supplies and its effect upon filters, and the effect of mine wastes and road tars upon water supply and water treatment.

While at the University of West Virginia Professor Carpenter not only impressed his dynamic personality upon his college students, but organized short courses for water works operators and firemen as extension work throughout the state in cooperation with various state agencies. He was productive in research, an excellent teacher, and continued occasional practice as a private consultant in sanitary engineering. His activities in the water works field led to his election as Chairman of the Central States Section and a member of the Board of Directors of the American Water Works Association.

In 1935 sanitary engineering instruction and research at the College of Engineering of New York University, in New York, N. Y., were undergoing rapid expansion. The largest sanitary engineering research laboratory connected with any educational institution in the United States had been devised under an agreement between the University and New York City. It became necessary, therefore, to put in charge of this development an outstanding sanitary engineering educator who combined practical experience with scientific knowledge, and who could direct large academic responsibilities along with research for industry and public agencies. A country-wide search indicated that Professor Carpenter was the ideal man for this novel undertaking. He was appointed associate professor in 1935, professor in 1936, and director of the Sanitary Engineering Research Laboratory in 1937.

During his five years in responsible charge of sanitary engineering at New York University Professor Carpenter brought to fruition the many facets of his strong personality. He developed an outstanding sanitary engineering department, both in undergraduate and graduate work. He took the chief part in the design and construction of the Sanitary Engineering Research Laboratory, managed and presented numerous important researches therein, sought out and added competent instructors to the teaching staff, organized short courses for state and municipal agencies, and was looked to as an outstanding authority in his field. He contributed alone or as co-author some ten important scientific articles to engineering publications, was active as chairman and member of various committees, and was generally regarded as one of the leading members of the faculty of New York University.

The esteem in which Professor Carpenter was held throughout the United States was evidenced by an invitation from the University of Southern California, in Los Angeles, to give a series of lectures in its Institute of Government in the summer of 1938. He was also a member of the Advisory Committee on Medicine and Public Health for the New York World's Fair 1939.

Shortly before his last illness, Professor Carpenter was requested to direct a very large and unique investigation on the land-fill method of garbage and refuse disposal. This method had never been scientifically studied, and the cooperative project organized by Professor Carpenter between New York University, the Department of Sanitation of New York City, and the Works Projects Administration was just beginning to produce results of great public importance at the period of his untimely death. It is continuing under different direction, but the results will undoubtedly serve as a crowning evidence of his wise counsel and creative ability.

Just before he had to relinquish participation in technical activities, Professor Carpenter had completed virtually a year's work on the organization and extensive editing of the "Manual of Water Quality and Treatment" issued by the American Water Works Association a few days before his death. This authoritative volume very appropriately carries a special acknowledgment of his important part in its preparation. It will remain for years to come as a fitting memorial of his contributions to water works practice and will be cherished by his countless friends and associates in this field of engineering.

No account of Lewis Van Carpenter's life and accomplishments should close without reference to his wife, Tempie (Amos) Carpenter, who survives him. The couple was married on March 17, 1923, at Fairmont, W. Va. Attending innumerable sanitary engineering gatherings with him in all parts of the country, she was nearly as well known to the "fraternity" as he was.

Professor Carpenter was a member of the American Water Works Association, Pennsylvania Sewage Works Operators Association, American Public Health Association, Society for the Promotion of Engineering Education, the New York State Sewage Works Association, and the New York State Society of Professional Engineers. He was a member of the honorary scientific and professional societies of Sigma Xi and Tau Beta Pi, and was an 18th degree Scottish Rite Mason.

The regard in which Professor Carpenter was held by the faculty of the College of Engineering at New York University is evidenced by this following excerpt from a faculty resolution:

> "A rare combination of scientific knowledge, teaching aptitude, and administrative ability, Professor Carpenter was beloved by students, admired and respected by faculty, sought after by industry and public agencies, and in constant demand for professional papers and professional society activities. As Chairman of the Joint Heights Faculty Committee on Plant he contributed greatly to wise and judicious improvements on our campus. Able, friendly, industrious, distinguished particularly by great integrity of character, Lewis Van Carpenter will remain long in the memory of the faculties at University Heights."

Professor Carpenter was elected an Associate Member of the American Society of Civil Engineers on November 15, 1926, and a Member on January 29, 1934.

ISAAC NEWTON CARTER, M. Am. Soc. C. E.[1]

Died April 8, 1940

Isaac Newton Carter was born at Bonham, Tex., on February 12, 1891, the son of I. M. and Rosa Ann (Coon) Carter. His early life was spent in the Southwest, particularly Texas and Oklahoma, and his secondary education was completed at Coronol Institute, San Marcos, Tex., in 1914.

Mr. Carter's professional training was divided among three institutions: 1916–1917 at Rice Institute, in Houston, Tex.; 1920–1923 at the University of Colorado, in Boulder, Colo.; and 1923–1925 at the University of Idaho, in Moscow, Idaho, where he received the degree of Bachelor of Science in Civil Engineering. Subsequently he received from the University of Idaho the degrees of Master of Science in Civil Engineering, in 1927, and Civil Engineer, in 1930.

Mr. Carter was essentially an engineering teacher, a job to which he applied himself with distinction, and to which he brought a background of real interest as well as actual experience in several phases of civil engineering. His teaching experience began as an assistant in engineering drawing in the vocational division, University of Colorado, and extended from 1921 to 1923. In 1923, as an assistant in engineering drawing, he began his long career at the University of Idaho. During his tenure he held every academic rank, including a full professorship which was attained on September 1, 1939. Although his specialty was engineering drawing, he also taught hydraulics, sanitary engineering, and municipal engineering. His work in engineering drawing culminated in the contribution of a notable textbook published in 1939.

Possessing an exceptional measure of human understanding, a sense of humor, and high personal and professional ideals, all born of broad experience gained on engineering projects, on the football field, and in academic halls, it was only natural that Mr. Carter became the respected counselor and adviser for all engineering freshmen at the University of Idaho, a service which he performed either officially or unofficially for fifteen years. In this capacity he left upon hundreds of embryo engineers that invisible yet indelible impression with which real teachers influence their students. He was affectionately known as "Ike" and was rated as a "square shooter," albeit firm and exacting. Moreover, he was selected as student chapter adviser and was called upon to serve on the University discipline committee.

Mr. Carter's professional record is recorded in *Who's Who in Engineering* for 1931 and in *American Men of Science*. Besides teaching, the record includes experience in reclamation and irrigation, sewage disposal, and special work in limnology in Glacier National Park, Montana. One activity which he promoted particularly was the program of short courses sponsored by the Pacific Northwest Section, American Water Works Association.

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[1] Memoir prepared by J. E. Buchanan, Assoc. M. Am. Soc. C. E.

Mr. Carter was active in civic, fraternal, and religious circles, having been a vestryman in the Episcopal Church, a Mason, a member of Acacia, and a member of the Moscow Chamber of Commerce.

His technical and professional affiliations included Idaho Society of Engineers, American Water Works Association, American Public Health Association, and the Society for the Promotion of Engineering Education. He was serving a term as vice-president of the Spokane Section of the Society at the time of his death.

On June 29, 1930, he was married to Louise Shaff Blomquist of Lewiston, Idaho, who, with one daughter, Dorothy, survives him.

He was a teacher, an engineer, a husband, a father, and a friend—faithful to his trusts and duties and respected as a man.

Mr. Carter was elected an Associate Member of the American Society of Civil Engineers on March 11, 1929, and a Member on February 13, 1940.

GEORGE WILLIAM CHICKERING, M. Am. Soc. C. E.[1]

Died March 14, 1940

George William Chickering was born in Lawrence, Mass., on December 11, 1869, the son of George Edward and Harriet A. (Barnes) Chickering. He was educated in the public schools of Lawrence, and later attended the Massachusetts Institute of Technology, at Boston, Mass., receiving a certificate in 1891 as a special student in the technical course.

From 1891 to 1892 he served as draftsman and assistant to the master mechanic in the Washington Mills in Lawrence; and during the illness of the latter, he served as acting master mechanic.

For the next two years he was head draftsman and designer for C. W. and C. P. Damon, Architects, of Haverhill, Mass. Then the firm of Chickering and O'Connell, Architects, was organized in Manchester, N. H., and Mr. Chickering spent the next eighteen years (1894 to 1912) as partner designer and superintendent of engineering. In 1912 he went into private practice as an architect and engineer, and operated his own business for five years.

During the World War Mr. Chickering served one year as engineer in the Construction Division of the U. S. Army, War Department, Washington, D. C., specializing on heating layouts.

From 1918 to 1926 he was partner, engineer, and specification writer in the firm of Peare, Quiner, Nevin and Chickering, Architects, in Boston. During these thirty-two years some forty school buildings, more than one hundred churches, and numerous other structures, most of them built in New England, were to the credit of Mr. Chickering. Among the finest of the ecclesiastical buildings was Trinity Parish Church at Newton Center, Mass., one of the most charming smaller churches in New England.

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[1] Memoir prepared by Charles K. B. Nevin, Newton Center, Mass.

From 1926 to 1927 he was a designer with the New York State Department of Architecture, at Albany, N. Y. Then he was resident engineer at the Country Club, at Waterbury, Conn., for which Parsons and Wait, of Boston, were the architects. For the four years from 1927 to 1931 he was with E. B. Badger and Sons Company, Boston, as architect and engineer.

From 1931 until his death he was designer with Harold R. Duffie, Architect, of Boston.

Mr. Chickering was a prominent and active member of the Masonic Order in Mt. Horeb Lodge, in Manchester, and Hiram Lodge, in Arlington, Mass.

In his engineering work he practiced, and demanded in others under him, a high degree of workmanship and skill. He was a man of unimpeachable integrity and fine character. He had a friendly disposition and cordial personality.

On June 12, 1895, Mr. Chickering was married to Maud P. Abbott, at Manchester. He is survived by his widow and three children—John A., Margaret E., and George E.

Mr. Chickering was elected a Member of the American Society of Civil Engineers on January 17, 1921.

CLARKE COPELAND COTTRELL, M. Am. Soc. C. E.[1]

Died August 21, 1940

Clarke Copeland Cottrell was born at Auburn, Nebr., on December 31, 1885, the son of the late Charles C. and Letitia J. (Copeland) Cottrell. His early education was obtained in the public schools of his native city, where he was graduated from high school in 1904.

During his senior year in high school, Mr. Cottrell got his first experience in civil engineering by assisting with the surveying and construction of five miles of macadam road near his home town.

Upon his graduation from high school, he taught school for one year and then registered in the College of Engineering, University of Nebraska, Lincoln, Nebr. While attending the university, he obtained practical experience in surveying and mapping by working on government surveys in the Black Hills of South Dakota.

In 1908, Mr. Cottrell was employed by a railroad company to survey a new line through the mountains of the State of Washington. Upon completion of this work in 1909, he organized the firm of Parrott, Cottrell and Parrott, consulting engineers with offices in Baker, Ore. This firm engaged in general civil engineering work, but specialized in highway and irrigation engineering in the states of Oregon and Washington.

Severing his connections with this firm in 1911, Mr. Cottrell was employed as engineer and superintendent of construction at Lyle townsite, Wash. He was in charge of the planning and construction of the townsite, including a water supply dam. As demands upon his time permitted, he maintained the private practice as consulting engineer.

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[1] Memoir prepared by H. J. Brunnier and T. E. Ferneau, Members, Am. Soc. C. E.

In July, 1913, due to his engineering achievements, he was encouraged by the U. S. Bureau of Public Roads to enter the employ of the California Highway Commission. As resident engineer and superintendent of day labor, he was in charge of the construction and maintenance of the state primary highway system in Marin, Sonoma, and Alameda counties.

He resigned his position with the California Highway Commission in June, 1917, and became assistant state highway engineer of Nevada, with a newly organized State Highway Department. On February 1, 1918, he was appointed state highway engineer and under his direction the primary organization of the Nevada Highway Department was completed. Also under his direction plans were made for the development of a state highway system, but State Highway Engineer Cottrell found that it was practically impossible to carry out these plans, as no funds were available to pay for the cost of construction. He observed that this same condition was prevalent in other sparsely populated western states.

Specifically, in Nevada more than 90% of the land was government-owned and the restrictions of the 1916 Federal Aid Act made it impossible for Nevada to qualify for government financial assistance in constructing its state highway system. This and other restrictions of the Act presented impassable barriers and, with the approval of the Governor of Nevada, Mr. Cottrell devoted practically all of his time to bringing these matters to the attention of the members of congress. To him it was a crusade, not only for Nevada but for all of the western states where such large areas of public lands existed. Utilizing every resource at his command, he interested civic organizations, automobile associations, and others in the fight to secure reasonable financial assistance from the "absentee landlord," the federal government, which was such a large property owner in this area. During the next two years he wrote thousands of letters and numerous magazine articles to awaken the public to the problem and, in turn, to bring the subject forcibly before the members of congress. His success as an organizer in this fight was outstanding.

In 1921 there was presented to congress the Phipps-Dowell Bill which provided that, of the 100% cost of construction of Federal Aid roads, the government would contribute 50% plus one half the percentage of public land within a particular state. Thus, if a state had 50% of its area still remaining as unappropriated public land, the government's share of the cost of construction would be 50% plus 25% for the public land within the state, or a total contribution of 75% of the cost of construction. Enactment of this legislation did not increase the amount of Federal Aid received by any of the public land states, but it did reduce the amounts which they were required to contribute toward the construction of federal-state highways. Congress approved this bill in October, 1921. The formula contained in this measure was evolved by Mr. Cottrell and was known as the "graduated scale" plan of Federal Aid. So practical was this basis of contribution that, in general, this method of assistance to western states still exists. Without it, undoubtedly, transcontinental highways across some of these states could not be as highly developed as they are today.

Visualizing a greater opportunity to continue the general work of good road development, Mr. Cottrell resigned as highway engineer for the state of Nevada in 1921 to accept appointment as manager of the Good Roads Bureau of the California State Automobile Association, with headquarters in San Francisco, Calif.

Private industry called him once more and during 1924 and 1925 he was chief engineer of the Western Willite Corporation, Los Angeles, Calif. Upon completion of the terms of his contract with that organization, he again became associated with the California State Automobile Association as manager of its Highway Bureau. In 1930 he also assumed charge of all public safety and legislative activities of the Association. These were later consolidated into the Public Relations Department of which he was the manager.

Almost two decades of service with the California State Automobile Association won for him a reputation throughout the entire West as an authority on public affairs involving the motorist. He was a leading figure in the development and support of measures and policies beneficial to the motoring public, and particularly was he effective in his opposition to proposals which were not to the best interests of the motorist. He contributed substantially to the progress of California as a motoring state and played a prominent part in placing the financing of California highways on the present "pay-as-you-go" basis. His leadership accounted largely for adoption of the highway expansion program by the 1933 session of the California Legislature, which added 6,600 miles of county roads to the state highway system. He also was a leader in successful efforts for allocation of reasonable shares of the gasoline tax to counties and municipalities; for the constitutional amendment prohibiting diversion of highway funds to non-highway purposes; for elimination of the personal property tax on motor vehicles and substitution of a state licence fee; and for many other measures which have benefited the motorist and contributed to the development of highways and the growth of motoring.

Traffic safety also commanded his attention, especially the need for uniform traffic laws and strict, impartial law enforcement. Under his direction the Automobile Association sponsored the School Safety Patrol and furthered safety instruction in the schools.

A keen student and an astute observer, he also possessed the ability to envision and plan comprehensive projects. His was a fine engineering mind—keen, analytical, and versatile. These qualities were demonstrated by his success in almost every field of civil engineering, culminating in advanced types of executive work requiring a well-rounded engineer as well as a man unafraid to pioneer, and with the initiative, energy, and courage to push plans to a successful conclusion.

Surviving him are his widow, Mrs. Maude E. Cottrell; two daughters, Lois (Mrs. Delbert M. Thompson) and Kathleen (Mrs. Walter Haluk); a brother, Roy M. Cottrell; and a half brother, Edgar Ferneau.

To his survivors he has bequeathed proud memories of a fine, full life of untiring activity and notable achievements. To his host of friends and fellow engineers, the great highway systems of the West, and particularly of California and Nevada, will continue to reflect, directly and indirectly, the ability, energy, and pioneering spirit of the man they knew and admired. To the remainder of mankind, he left what all thinking men strive to leave behind—a few blocks, unselfishly added to the structure of progressive civilization, leaving the world a better place for his having passed through it.

He was a Registered Professional Civil Engineer of the State of California; a member of the Masonic Fraternity and of Islam Temple of the Mystic Shrine. His recreation was taken mainly at the Lake Merced Golf and Country Club at San Francisco, of which he was a member.

Mr. Cottrell was elected an Associate Member of the American Society of Civil Engineers on May 13, 1918, and a Member on September 9, 1919.

FRED GASTON CUNNINGHAM, M. Am. Soc. C. E.[1]

Died January 26, 1941

Fred Gaston Cunningham was born at Bellaire, Ohio, on March 31, 1886, the son of William Nash Cunningham and Virginia (Gerby) Cunningham. He received his early education in the public schools of Toledo, Ohio.

During the period from 1903 to 1907, Mr. Cunningham served as a rodman, instrumentman, and draftsman on railroad and municipal engineering work in the Middle West. From 1907 to 1909, he was engaged as chief draftsman for the Fort Worth and Denver City Railroad Company at Fort Worth, Tex., and as assistant engineer on the design and construction of the Toledo municipal water treatment plant. During this period he became greatly interested in municipal engineering problems and spent his spare time in extensive study. In 1909, he entered the University of Pennsylvania at Philadelphia, Pa., as a special student and concentrated for the next two years on courses pertaining to sanitary engineering.

From 1911 to 1914, Mr. Cunningham was resident engineer and later chief draftsman for W. J. Sherman Company, Consulting Engineers of Toledo, and for that firm he had charge of more than twenty water supply, sewerage, and paving projects in Ohio and neighboring states. From 1914 to 1921, he was first principal assistant engineer and later associate consulting engineer with William G. Clark, Jun. Am. Soc. C. E., of Toledo and was in general charge of the design and construction of water supply structures, pumping stations, and supply mains for Toledo, Lima, and Fremont, Ohio, and Alpena, Mich.

In 1921, Mr. Cunningham joined the staff of Fuller and McClintock, Consulting Engineers, specializing in sanitary engineering work. He served as a principal staff member until 1928, when he became a partner of the late

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[1] Memoir prepared by Ernest W. Whitlock, M. Am. Soc. C. E.

George W. Fuller,[2] M. Am. Soc. C. E., and the late James R. McClintock,[3] M. Am. Soc. C. E. After Mr. Fuller's death in 1934, Mr. Cunningham continued the consulting engineering practice of Fuller and McClintock as the sole surviving partner.

Although a considerable part of Mr. Cunningham's experience was related to water-works pumping stations and equipment, he had broad and diversified experience in the engineering of water works and sewage treatment works. From 1921 to 1924 he had charge of an investigation and report, followed by design and construction of a new $3,000,000 water supply works for Memphis, Tenn., which included wells, filters, pumping stations, and reservoirs. From 1930 to 1933 he had charge of the design and construction of a similar additional supply project for Memphis. Other work of Fuller and McClintock in which he took an active and responsible part included the design of the complete new water supply system for Kansas City, Mo.; the design and construction of aqueducts, pumping facilities, storage reservoir, and the control works of the Wanaque System of the North Jersey District Water Supply Commission; design of a new filter plant, pumping station, and pipe lines for Dallas, Tex.; design and construction of new pumping facilities, pipe lines, additional filters and water softening works for Clarksburg, W. Va.; design of the Wards Island Sewage Treatment Plant for the City of New York, N. Y.; design and construction of the Rahway (N. J.) Valley joint sewage treatment works; the design and construction of the New Britain, Conn., sewage treatment plant, and the design of the Bostwick Avenue Treatment Works in Bridgeport, Conn. Mr. Cunningham was engaged in supervising the construction of the last-named project at the time of his death.

Mr. Cunningham's keen mind and his capacity for sound analysis and clear thinking resulted in his being selected for a number of important reports, appraisal and valuation assignments. During his last years, the late George W. Fuller depended to a large extent upon Mr. Cunningham's judgment and capacity for sound analysis of engineering problems. In 1929, Mr. Fuller sent him to Europe to inspect and study sanitary engineering methods and structures.

Mr. Cunningham was a member of the American Institute of Consulting Engineers, of which he served as a past member of the governing council and as a past chairman of the committee on admissions. He was also a member of the New Jersey Sewerage Works Association and the American Water Works Association. In the latter organization he was a director from 1931 to 1935, past president of the New York Section, past chairman of the Publications Committee, and a member of the Waterworks Practice Committee. Mr. Cunningham was a Mason. During the World War he served as First Lieutenant in the Engineering Corps.

Mr. Cunningham was married to Edith Gempel on June 10, 1922. He is survived by his widow and two sisters, Dr. Bess V. Cunningham and Mrs. Lyle S. Hubbard.

Mr. Cunningham was elected an Associate Member of the American Society of Civil Engineers on March 7, 1921, and a Member on July 12, 1926.

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[2] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 100 (1935), p. 1653.

[3] For memoir, *loc. cit.*, p. 1693.

WILLIAM HENRY DITTOE, M. Am. Soc. C. E.[1]

Died May 27, 1939

William Henry Dittoe was born at Somerset, Ohio, on November 14, 1885. He was the son of Frank A. and Mary (Finck) Dittoe. When Mr. Dittoe was but one year old the family moved to Columbus, Ohio. He received his early education at Central High School in Columbus. Upon graduation from high school he enrolled at Ohio State University, in the same city, from which he was graduated in 1908 with the degree of Ceramic Engineer. He was a member of Sigma Chi fraternity, Theta Nu Epsilon engineering honorary fraternity, and was one of the charter members of both Bucket and Dipper, and Sphinx, junior and senior honor societies, organized for recognition of prominence in school activities.

Mr. Dittoe evinced an early interest in sanitary engineering. In the summer and fall of 1905 he was employed as inspector at the Columbus Sewage Testing Station, at the time one of the outstanding endeavors in sewage treatment research. During the summers of 1907 and 1908 he was in charge of the operation of the water purification and sewage treatment plants at Camp Perry, Ohio, a National Guard encampment on the shore of Lake Erie.

In October, 1908, Mr. Dittoe entered the employ of the Ohio State Board of Health (later the Ohio Department of Health) and remained with that organization, except for a brief period during the World War, until April, 1926. His first engagement with the State Board of Health was in the capacity of assistant engineer. In December, 1911, he was appointed acting chief engineer, and in February, 1912, received the appointment of chief engineer, remaining in that capacity until he left the Department of Health in April, 1926. In 1918 he received a commission as captain in the Sanitary Corps, and was in training at Camp Greenleaf in Chickamauga Park, Georgia, at the time of the Armistice in November, 1918.

During Mr. Dittoe's regime with the Ohio Department of Health the engineering division of the department attained a high standing among the corresponding divisions of other state health departments. Mr. Dittoe was recognized nationally as a leader in health department activities with respect to water supply, sewerage, and general sanitation. He was active in the sanitary engineering section of the American Public Health Association, the American Water Works Association, and the American Society of Civil Engineers, serving on various committees and as an officer. He was also a member of the American Society of Mechanical Engineers. He served as a chairman of the sanitary engineering section of the American Public Health Association and as a chairman of the sanitary engineering section of the Society. He was most active in the Central Ohio Section of the Society, being president in 1924.

Mr. Dittoe was a vital force in the enactment of legislative statutes in Ohio having to do with sanitary engineering measures. While with the Department of Health he assisted in drafting statutes which delegated to the Department of Health additional authority over that already granted in earlier years to require improvements in water supply, sewage disposal, and stream pollution correction. In addition he initiated and assisted in drafting statutes authorizing the establishment of Sanitary Districts in Ohio which permit a single authority in areas of overlapping political subdivisions for the purpose of installing water supply and sewerage improvements. He was also instrumental in having enacted the so-called "sewer rental law" in 1923, permitting municipalities to charge for sewerage service and thereby derive funds for the maintenance and operation of sewers, sewage treatment, and disposal works. In 1934 he was appointed by the Governor to serve on a committee of five to recommend recodification of the state laws governing conservancy and sanitary districts; these recommendations were accepted by the Ohio General Assembly by enactment of amendments to the General Code adopted in 1937.

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[1] Memoir prepared by a Committee of the Central Ohio Section, consisting of W. H. Knox, F. D. Stewart, and F. H. Waring, Members, Am. Soc. C. E.

In April, 1926, Mr. Dittoe accepted appointment as chief engineer of the newly formed Mahoning Valley Sanitary District, which was organized to furnish a new water supply for the cities of Youngstown and Niles, Ohio. This work included the construction of a reservoir and a dam impounding ten billion gallons of water, a 40-mgd water purification and softening plant, pumping station, supply mains from the pumping station to Youngstown and Niles, and distributing reservoirs. The total cost of these projects was about $9,000,000. After completion of the construction of these projects Mr. Dittoe continued as secretary and chief engineer of the Mahoning Valley Sanitary District until his death.

Mr. Dittoe was admired and respected by all who knew him, not only for his recognized technical knowledge and administrative ability, but for his high character and integrity and his human and lovable nature. The sanitary engineering profession as a whole is deeply indebted to him, for he contributed no small part in the advancement of the sanitary engineering science during his professional life.

He was married on October 20, 1912, to Helen Marie Spencer of Columbus, who died on February 10, 1920. Two children of his first marriage, William Edward and Mary Elizabeth (Mrs. Franklin Ward) survive him. On April 2, 1924, he was married to Marie Leslie of Columbus and is survived by his widow and three children of his second marriage—Charles Leslie, Eugenia, and Frank Allen. He is also survived by a sister, Mrs. Walter Cott. His mother survived him at the time of his death but died in February, 1940.

Mr. Dittoe was a member of the Youngstown Chamber of Commerce and was active in the promotional efforts of that organization to obtain recognition of the federal government for navigation improvements comprising canalization of the Beaver and Mahoning rivers. He was past-president of the Mahoning Valley Society of Professional Engineers. He was a member of Hillman Lodge, Free and Accepted Masons, of Youngstown. He belonged to the First Presbyterian Church of Youngstown.

Mr. Dittoe was elected a Junior of the American Society of Civil Engineers on May 3, 1910; an Associate Member on October 1, 1912; and a Member on September 9, 1919.

GEORGE GOODELL EARL, M. Am. Soc. C. E.[1]

Died September 16, 1940

George Goodell Earl was born on October 9, 1863, in Monmouth County, New Jersey. He was the only child of Holmes and Anna (Taylor) Earl. The Earl family was one of the oldest New England families. His progenitors, Ralph and Ione Earl, emigrated from England and settled in Portsmouth, R. I., in 1628.

His early education was obtained in the Freehold Institute, in Monmouth County. He later attended Lafayette College at Easton, Pa., where he studied Civil Engineering and was graduated in 1884, receiving the degree of Civil Engineer. In 1918, in recognition of the signal contributions he had made in the field of municipal sanitation, his alma mater conferred on him the degree of Doctor of Science.

After leaving college he was employed, during 1884 and 1885, by the U. S. Geological Survey in New Jersey. For the three years following, his activities were in the field of railroad engineering, he having been employed, from 1885 until March, 1886, as levelman by the Baltimore and Ohio Railroad Company, and subsequently, from April, 1886, to March, 1888, by the Atchison, Topeka, and Santa Fe Railway Company in the various capacities of topographer, transitman on location, and as division engineer on construction.

A decline in railroad construction and extension, at about this time, making continuous employment more or less uncertain, together with the fact that the future seemed to hold brighter prospects in the field of municipal engineering, induced him to make a change in his previous line of endeavor.

From April, 1888, to May, 1889, he was assistant engineer on sewerage construction, in Montgomery, Ala. From May, 1889, to March, 1891, he was a member of the firm of Williamson and Earl, in Montgomery, practicing municipal engineering, during which time he was in charge of planning and constructing, simultaneously, sewerage systems in the two neighboring towns of Cordele, Ga., and Americus, Ga., for a period of about one year. Later he served for one year as city engineer of Americus. From March, 1891, to July, 1892, he was engaged in street railway work and suburban development at Montgomery.

In July, 1892, he came to New Orleans, La., where he was destined, through a long and successful career, to achieve an eminence in sanitary engineering seldom attained by members of the engineering profession. His opportunity was great, but his responsibilities were greater. The City of New Orleans, in 1892, with a population of 280,000, was a city built on a flat terrain of soggy soil, disposing of its sewage in the most primitive manner, and obtaining its drinking water supply from the runoff of roofs which was stored in cypress cisterns on individual premises. Fortunately, excessive drought conditions were averted by an evenly distributed annual rainfall of about 60 in.

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[1] Memoir prepared by a Committee of the Louisiana Section composed of John Klorer, John T. Eastwood, and W. B. Gregory, Members, Am. Soc. C. E.

The first serious attempt on the part of the city government to ameliorate these conditions came with the granting of a franchise to the New Orleans Sewerage Company for the construction of 140 miles of sewers in five years. George Earl was selected as chief engineer of this company, to make the necessary surveys, designs, and estimates. In 1895, the company encountered financial difficulties, and, with the work partly completed, it went into the hands of a receiver. The court ordered the receiver to retain the services of Mr. Earl.

In 1899, the citizen property owners of New Orleans held an election and voted a special tax for sewerage and water supply system which was to be municipally owned and operated by an agency of the city and designated as the Sewerage and Water Board of New Orleans. In 1900, upon the sale of the necessary bonds secured by this tax, Mr. Earl was designated by this board as its chief engineer and superintendent. Two years thereafter, it was decided, by proper legislation, to merge and include the activities of the New Orleans Drainage Commission within the jurisdiction of the Sewerage and Water Board, and these were also placed under the direction and supervision of Mr. Earl.

For thirty-one years he discharged the duties assigned to him with a fidelity to public interest that won the high regard and respect of the citizens of the community in which he lived. He was looked upon as the father of the sewerage and water systems of New Orleans, concerning which little or nothing had been accomplished before he took hold—at least nothing that was worth being incorporated in the present existing systems. In drainage improvement, although it is true that a well-defined plan had been adopted and a definite start made antedating Mr. Earl's incumbency, still he was largely responsible for the extensions and improvements made necessary by the progress and development of the city after he became identified with this important utility. All told, something like fifty-one millions of dollars were expended in construction work, under his direction, on the sewerage, water, and drainage systems of the City of New Orleans.

Occasionally Mr. Earl would venture into side lines of his profession and demonstrated that he possessed mechanical ingenuity to a high degree. Several devices pertaining to automatic regulation and measurement of fluid pressures and flow were created and perfected by him and are in use at the New Orleans Water Purification Plant.

In 1931, he retired from his active duties, but was retained as consulting engineer by the Sewerage and Water Board until the date of his passing.

The following editorial expression of one of the New Orleans newspapers on his accomplishments is a well-deserved tribute:

> "His work was of the highest form of creation, and had an enormous influence upon the destiny of this city; for New Orleans could not have made the progress which distinguished it since the creation of the Sewerage and Water Board, in 1900, without these facilities. The problems in New Orleans were different from those posed by any other large center in this country.

> "He planned well, he built well, he administered well. In a way, the modern New Orleans is a monument to his memory. He took an active interest in all civic developments. He was respected, he was liked, he was loved. He holds a high place in the history of our City."

He was a Life Member of the Louisiana Engineering Society, and a past-president of the American Water Works Association (1915); he was also a member of the American Public Works Association, and the New Orleans Association of Commerce.

One of the honors accorded him, which he prized greatly, was his appointment, in 1924, as a member of the Chicago Engineering Board of Review on the Chicago Diversion of Water from Lake Michigan. As a result of the information gained while a member of this committee, he became a strong advocate in opposition to the St. Lawrence Waterway Treaty, and defended his position before congressional committees and in papers read before local engineering bodies.

Mr. Earl was married, in 1890, to Anna L. Riddell, of Rothville, Mo., who died on May 24, 1911. The issue of this marriage was a son, Ralph, who has followed the profession of his father. In 1912, he was married to Frances H. Fowler, of New Orleans, and a son of this marriage, Thomas Collins, likewise has followed the engineering profession. The aforementioned, and three grandchildren, are the only surviving near relatives.

Mr. Earl was elected a Junior of the American Society of Civil Engineers on May 7, 1890; an Associate Member on December 2, 1891; and a Member on April 4, 1894.

SAMUEL ECKELS, M. Am. Soc. C. E.[1]

DIED SEPTEMBER 23, 1939

Samuel Eckels was born in West Brownsville, Pa., on January 22, 1884, the son of Charles E. and Louisa (McCrory) Eckels. He received his early education in the public schools of South Brownsville and was graduated from the high school of that city. Upon completion of his high school course, he entered Washington and Jefferson College, at Washington, Pa., receiving the degree of Bachelor of Science in 1905.

During school vacations he was employed by the River Coal Company, in Brownsville, Pa., surveying, and inspecting concrete and steel construction. After graduation from Washington and Jefferson College, he spent one year with the Pennsylvania Railroad Company as instrumentman. In April, 1906, he obtained a position with the City of Pittsburgh, Pa., where he was employed continuously for eighteen years, with the exception of a period of service in the first World War. In October, 1918, the Secretary of the Navy assigned him to the construction of recreation centers for the U. S. Marines in Santo

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[1] Memoir prepared by E. H. Stumpf, Assoc. M. Am. Soc. C. E., and George W. Schusler, M. Am. Soc. C. E.

Domingo, Cuba, and Porto Rico. He served for six months after the Armistice.

While with the City of Pittsburgh he served in the following capacities: Construction engineer of the Filtration Plant at Aspinwall, Pa.; engineer in charge of all City Park improvements; division engineer in charge of the Division of Public Utilities looking after the city's interest in the use of city streets by the Public Utility Corporation; and finally, assistant chief engineer of the Bureau of Engineering.

In 1924 Mr. Eckels was made assistant chief engineer of the Allegheny County Road Bureau, and in 1925 he was made chief engineer in charge of design and construction. During his engagement with the County of Allegheny, he had charge of many notable improvements, eliminating many so-called "bottlenecks."

In 1927 he was appointed chief engineer of the Pennsylvania Department of Highways, and was stationed at Harrisburg, Pa. He served in this capacity for eight years, and commendable road improvements were made during his term of office. He was regarded as the ranking engineer of highway design, and represented the United States at the International Road Congress held in Washington, D. C. While with the Department of Highways, he constructed many rural road systems styled "Pinchot Roads."

Mr. Eckels was appointed chief engineer of the Allegheny County Authority in April, 1935, and successfully planned and constructed many large improvements, notably the Pittsburgh-Homestead High Level Bridge and approaches.

At the time of his death, he was in private practice as a consulting engineer with offices in the Grant Building, in Pittsburgh. He had been retained by many large corporations, notably the Pennsylvania Railroad Company in its litigation with the City of Pittsburgh, his testimony leading to a recovery of more than $800,000 for damages sustained.

Mr. Eckels served as president of the American Society of Municipal Engineers, president of the Pittsburgh Chapter of the Pennsylvania Society of Registered Professional Engineers, chairman of the Convention Committee of the National Society of Professional Engineers, and vice-president of the American Road Builders Association. He was a member of the American Society of Testing Materials, and held an official connection with the American Association of State Highway Officials for four years, during which time (1930) he was not only president, but host to the annual meeting which met that year in Pittsburgh. He was a member of the Mt. Lebanon Methodist Church, was past-master of Brownsville Lodge No. 60, Free and Accepted Masons, and later became master of Roosevelt Lodge No. 697 of Pittsburgh and served as secretary of this Lodge for several years. He was a member of all the masonic bodies, and of the St. Clair Country Club, in Pittsburgh.

It is not easy to pick out the most important of Mr. Eckels' many striking characteristics, but certainly a most outstanding one was his absolute and unfailing intellectual integrity. The unusual quality of his fine, rare intelligence and power of initiative is shown by his many achievements, not only in the engineering field, but in civic matters. He had a rare gift of friendship and accepted people as they were, always seeing the best in them. It is difficult to imagine a finer companion than he was, either as a business associate or as a fellow sportsman, and he lived his life to the full. His administrative qualifications seemed to be unlimited. Having a pleasing and striking personality, he was a leader in the profession during the many capacities in which he served. His sudden and untimely death comes as a personal loss to his many friends all over the United States. He was buried in Bridgeport Cemetery, in Brownsville.

He was married on March 24, 1909, to Carrie Waggoner of Brownsville, who, with a daughter, Eleanor Eckels, and a son, Lee W. Eckels, survives him. He also left, as immediate members of his family, a brother, George A.; and two sisters, Elizabeth and Fanny (Mrs. Lewis Lewis).

Mr. Eckels was elected an Associate Member of the American Society of Civil Engineers on October 14, 1919, and a Member on March 15, 1926.

GEORGE STOKES FANNING, M. Am. Soc. C. E.[1]

DIED JANUARY 2, 1941

George Stokes Fanning was born in Detroit, Mich., on April 25, 1885, the son of Charles A. and Carrie (Stokes) Fanning. He attended the University of Michigan, at Ann Arbor, Mich., and was graduated with the degree of Bachelor of Science in Civil Engineering in 1906.

Mr. Fanning entered railroad service in 1906 as a rodman for the Michigan Central Railroad Company. He served that company until 1907. From then until 1910 he was instrumentman and assistant engineer for the Detroit River Tunnel Company. From 1910 to 1913 he was resident engineer of the Algoma Central and Hudson Bay Railroad Company. In 1913 Mr. Fanning entered the service of the Erie Railroad Company and served as follows: 1913–1914 as resident engineer; 1914–1916 as estimating engineer; 1916–1918 as chief draftsman; 1918–1920 as assistant to chief engineer; 1920–1925 as office engineer; 1925–1927 as principal assistant engineer; 1927–1929 as assistant chief engineer; and from 1929 until his death as chief engineer. His office as chief engineer of the Erie Railroad Company was at Cleveland, Ohio.

Mr. Fanning was a well-balanced engineer; his practical knowledge combined with his technical training and his pleasing personality placed him at the top of his profession where he commanded and had the respect of all of his colleagues. He was energetic in presenting new ideas, and constantly had in mind the bettering of the standing of the engineering profession. His presentations were always made in a very forceful manner and his disarming frankness was of great help to him in all of his contacts with his superiors, associates, and public officials. He possessed to a marked degree the faculty of inspiring confidence and was in all cases most insistent in obtaining definite results. He had a fine ability to grasp a situation quickly and make a prompt decision. He devoted his time to the main essentials of matters before him and was willing to leave the details to his subordinates.

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[1] Memoir prepared by E. M. Hastings, M. Am. Soc. C. E.

Mr. Fanning's entire career was devoted to railway service. He was a member of the American Railway Engineering Association and devoted considerable time and interest to that association. He was a director from 1933 to 1936; vice-president from 1938 to 1940; and president from 1940 until his death.

He was a member of the University of Michigan Club, a member of the Cleveland Chamber of Commerce, and a member of the "Cleveland Roamers," a golf organization in Cleveland and vicinity. He derived much pleasure from his contacts in this latter organization. At one time he belonged to the Canterbury Country Club in Cleveland, resigning about 1939 after he had acquired a farm in Pennsylvania. Farming developed into quite a hobby for him, and he spent most of his available spare time there deriving relaxation and enjoyment. It was his ambition, upon retirement from active life in the engineering field, to take up his home on his Pennsylvania farm and spend his closing years as a gentleman farmer.

Mr. Fanning was quite a reader on topics of general interest in addition to the engineering literature. He possessed a talent not known generally to many of his associates—a natural talent for art and music. His sketches of scenes and objects that he saw which interested him were of considerable merit. As a young man he played the piano very well and he was not too busy to continue playing through his later years. He was known to many stamp collectors for his unusual collection of first issues of the world which he displayed on occasion at various clubs.

Mr. Fanning was deeply interested in young men, particularly young men entering the engineering profession. His counsel and affectionate interest have left their impression on a number of young men, and have been of great assistance in guiding them through difficult problems.

In the engineering profession Mr. Fanning was best known to the engineering officers of the railways and was admired greatly for his professional attainments; his opinions and judgment were respected by all who knew him. Those who knew Mr. Fanning best admired him most for those fine qualities of character, moral integrity, high and noble purposes, and for his love of home and family. He was the type of engineer and gentleman who attains the summit of his profession and whom the profession delights to honor.

On June 18, 1914, Mr. Fanning was married to Nola M. Bell of Saegerstown, Pa. They had one daughter, Janet A., who, with Mrs. Fanning, survives him.

Mr. Fanning was elected a Member of the American Society of Civil Engineers on December 9, 1940.

FREDERICK ELBERT FIELD, M. Am. Soc. C. E.[1]

Died January 27, 1941

Frederick Elbert Field, the son of David C. G. Field and Lucy Ann (Hayden) Field, was born at Montague City, Mass., on November 16, 1873. His father served with distinction on the staff of Gen. Benjamin F. Butler during the Civil War and later became a member of the firm of Cutler, Lyons and Field, manufacturers, of Greenfield, Mass.

Mr. Field spent his boyhood in Montague City and received his early education at the Grammar School in Lowell, Mass., the High School at Northampton, Mass., and at Williston Seminary of Northampton College. He then entered the Massachusetts Institute of Technology, at Boston, Mass., from which he was graduated in 1896 with the degree of Bachelor of Science in Sanitary Engineering.

He was at once appointed assistant engineer in the Department of Water Supply of Boston, Mass., and during the next five years not only supervised the installation of a cast-iron pipe line under the harbor between Boston and South Boston but also acquired valuable experience in the design, construction, and operation of the water distribution system of that city. He early attracted the favorable attention of his superiors by reason of his tireless energy, unquestioned integrity, mastery of detail, and conscientious performance of all work for which he was responsible, and thus laid the foundation for future success.

In 1901 Mr. Field moved to Philadelphia, Pa., where for the next two years he served as assistant engineer of the Bureau of Filtration on the design and construction of the Upper Roxborough filtration plant. His work on this important project attracted the favorable attention of several prominent engineers with whom he was to be closely associated in subsequent years.

He left Philadelphia in 1903 to accept a responsible position with the well-known firm of Chapin and Knowles, Consulting Engineers, of Pittsburgh, Pa. He assisted in the design of the filtration plant for Lawrence, Mass., and was in charge of the design and construction of the water supply system for Berlin, Pa.

When the City of Pittsburgh organized its Bureau of Filtration in 1904, Mr. Field was appointed division engineer on the filtration plant at Aspinwall, Pa. There he supervised the construction of the sedimentation basins, administration building, and fifty-six slow sand filters, each one acre in area, that constituted the principal features of this important development in water purification.

Upon the completion of the Aspinwall plant in 1910, Mr. Field joined the Pittsburgh Flood Commission as assistant engineer. Also he became a member in the firm of Field, Barker and Underwood, Contractors, of Philadelphia, which connection was later terminated because of other responsibilities.

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[1] Memoir prepared by Frank Y. Dorrance, Assoc. M. Am. Soc. C. E.

In April, 1910, the City of Montreal, Que., Canada, commissioned the late Rudolph Hering[2] and George W. Fuller,[3] Members, Am. Soc. C. E., as consulting engineers, to report upon investigations into the best means of securing an improved water supply for Montreal, the largest city in Canada and one of the most important commercial and shipping centers in North America. They recommended that the water supply be taken from the St. Lawrence River, brought by gravity flow through an open canal to the city's water works in Verdun, Que., and then filtered in a plant of the double filtration type, having an initial capacity of 50, and an ultimate capacity of 150, million Imperial gallons per day.

Mr. Field was engaged by the consulting engineers to prepare the detailed designs and the specifications for the final filters in this project, and in 1911 he was appointed by the City of Montreal as resident engineer for the filtration works. Thus he entered the special branch of engineering to which he devoted the remainder of his years and in which he was recognized as an authority by his fellow engineers throughout Canada and the United States.

Mr. Field supervised the construction of the original Filtration Works, which included a low-lift pumping station, 16 pre-filters of the mechanical type, a like number of final or slow sand filters equipped with mechanical washing machines, a large covered reservoir for storage of filtered water, also the necessary chlorination apparatus. The exceptionally high quality of workmanship in evidence throughout this plant reflects the careful and thorough supervision he gave to every detail of construction.

On December 25, 1913, a serious break occurred in the conduit supplying the Low Level Pumping Station and interrupted the water supply for a period of nine days. Following this disaster a Board of Engineers recommended that an emergency system of water supply be built from the Lachine Canal to the pumping station, which work was designed and completed in record time under the direction of Mr. Field. The necessity and value of this emergency supply were proved in July, 1914, when the Lateral Conduit had to be closed for repair.

In 1915 Mr. Field became resident engineer on the Filtration and Aqueduct Works, which dual position he occupied until 1919 when he was appointed assistant superintendent of the Water Works Department.

After the completion of the Filtration Works, Mr. Field devoted most of his attention to the Aqueduct, or open canal, then under construction from the St. Lawrence River to Verdun for water supply and power development purposes. Later he rendered valuable services to the city in connection with several law suits which followed the decision of the authorities to discontinue work and cancel existing contracts, pending further investigation of the project, which subsequently was materially modified and completed by the Montreal Water Board.

Mr. Field served as consulting engineer for the City of St. Hyacinthe, Que., Canada, in connection with the design and construction of a filter plant, pumping station, and other additions to the water works system.

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[2] For memoir, see *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 1350.

[3] For memoir, *ibid.*, Vol. 100 (1935), p. 1653.

Upon the organization of the Montreal Water Board in May, 1920, Mr. Field was appointed filtration engineer in charge of the extension of the Filtration Works to its present capacity of 150 million Imperial gallons per day and other major improvements to the water supply system of Montreal. He planned the extensive park development for the large area surrounding the filtration works and the low-level pumping station that forms an attractive feature of the water works property along Joseph Saint Verdun. He made extended studies for a proposed 300 million gallon reservoir in the eastern section of Montreal and for other important projects which have had to be deferred because of the war.

Mr. Field was a member of the American Water Works Association, and of the Corporation of Professional Engineers of Quebec. For many years he was a member of the Engineering Institute of Canada. To these organizations and to various technical publications he contributed a number of papers and articles upon the Montreal Water Works System, the construction and the operation of the Montreal Filtration Works, and upon problems of water purification and chlorination.

Mr. Field was a member of the Summerlea Golf Club, also a member and Past-President of the Montreal West Curling Club, and he enthusiastically participated in these sports for many years. He attended the Montreal West United Church.

He was married in Greenfield on June 18, 1902, to Edith B. Newcomb who died on December 7, 1936, in consequence of a fire that accidentally developed in a house where she and Mr. Field were visiting at the time. This tragic experience seriously affected Mr. Field and was undoubtedly responsible for a gradual decline of health which continued for four years and culminated in his death on January 27, 1941, at the age of sixty-seven years.

He is survived by a son, Charles N. Field, a daughter, Margaret (Mrs. H. Aird Boswell), and four grandchildren; also by a sister, Bertha Field.

Mr. Field was exceptionally conscientious in his work and was respected by his associates and friends because of his ability, integrity, and fundamentally sound principles of conduct. He was a credit to the engineering profession.

Mr. Field was elected an Associate Member of the American Society of Civil Engineers on October 7, 1903, and a Member on September 6, 1910.

GILBERT HAVEN FRIEND, M. Am. Soc. C. E.[1]

DIED JANUARY 25, 1940

Gilbert Haven Friend, the son of David Adolphus and Phoebe (Rush) Friend, was born at Friendsville, Md., on February 5, 1882. He was the direct descendant of Nicholas Friend, a sea captain of Weymouth, England, who, with his five sons and five daughters, immigrated to America in 1675 and settled near Chester, Pa.

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[1] Memoir prepared by Thomas R. Beeman, M. Am. Soc. C. E.

Gilbert Friend's immediate ancestors were the earliest settlers of Garrett County, Maryland, having moved to what is now the town of Friendsville in 1778. His father was an active member of the West Virginia Conference of the Methodist Episcopal Church from 1883 to 1916 and continued his ministry until his death in 1926. Thus, there was early implanted in Gilbert Friend the deep religious convictions which influenced him throughout his life.

After receiving his elementary education in schools of Maryland and West Virginia, Mr. Friend attended the West Virginia Conference Seminary (later West Virginia Wesleyan) at Buckhannon, W. Va., taking a scientific course. He was graduated in 1901 with the degree of Bachelor of Science.

Following his graduation he served successively as field assistant, U. S. Geological Survey; topographer for the Coal and Coke Railway Company on railroad location surveys; and engineer for S. D. Brady and Brothers, Clarksburg, W. Va., engaged in mine surveying; layout of new coal mines, tipples, and appurtenant works; and location and construction of railroad sidings and branch lines to serve new mines.

In 1903, at the age of twenty one, Gilbert Friend, the young engineer, entered the employ of the Western Maryland Railway Company to follow what proved to be his main vocation and interest in life—railroad engineering. He remained in the employ of the Western Maryland Railway Company for four years, serving successively as instrumentman on railroad surveys, and resident engineer on railroad construction.

In 1907 Mr. Friend transferred his activities to the Far West, serving from 1907 to 1909 as assistant engineer with the Western Pacific Railroad Company, in charge of constructing a 40-mile division of that railroad in Nevada, and from 1909 to 1910 as locating engineer with the Oregon-Washington Railroad and Navigation Company in charge of locating 25 miles of railroad along the Deschutes River in Oregon.

He returned to the Western Maryland Railway Company in 1910, serving until 1913 as principal assistant engineer in charge of location and construction of 93 miles of railroad between Cumberland, Md., and Connellsville, Pa., in addition to extensive line-revision work and construction of new passenger, freight, and terminal facilities at Cumberland. During the following twelve years he was successively division engineer and assistant chief engineer of the Western Maryland Railway Company, in charge of maintenance of way on 650 miles of railway line. From 1925 to 1929 he was special assistant to the chief engineer in charge of the design and construction of McComas Street Terminal in Baltimore, Md.

Leaving the employ of the railway company in 1929, he became director, treasurer, and general manager of the Silica Tile Company, at Washington, D. C. From 1930 to 1934, he was successively engineer and senior engineer, civilian assistant, to the construction quartermaster, U. S. Army, engaged in the construction of army quarters at Fort Humphreys, Va., and the layout and construction of New Bolling Field (airport) at Washington, D. C.

Mr. Friend was employed by the U. S. Engineer Office at Zanesville, Ohio, in May, 1934, as senior engineer (civil). He served successively as chief of the railroad section engaged in the relocation and construction of 51 miles of railroad in flood-control reservoirs, and as chief of the Engineering Division, engaged in completing the construction of fourteen flood-control dams.

In August, 1938, he was transferred to the U. S. Engineer Office, at Pittsburgh, Pa., where, as senior engineer (civil), he organized and served until his death as head of the Railroad-Public Utility-Highway Section, engaged in the relocation of important facilities in projects of the Pittsburgh flood-control program.

Gilbert Friend was widely recognized as an outstanding railroad engineer and, while assigned to the Pittsburgh District of the U. S. Engineer Department, his services were frequently requisitioned by the chief of engineers, U. S. Army, as consultant on railroad work in other engineer districts of the United States.

Mr. Friend possessed a forceful character and was a natural leader of men. He was a capable executive, and, while he demanded of his subordinates as well as of himself the utmost of effort and attention to duty, he was eminently fair in all of his contacts and held to an unusual degree the respect, loyalty, and affection of his associates and subordinates. His mind was alert to the latest developments in his profession. He was a good student, and in his spare time he pursued a law course at the National University, in Washington, D. C. In 1928 he received the degree of Bachelor of Laws from that university. His ample and well-read library contained volumes on a variety of subjects. He was a student of history, and was particularly well informed on the early colonial history of the United States.

He was a delightful companion, and his many friends realized that under a rather stern exterior he masked a warm, charitable, and sympathetic heart. His interest in and helpfulness to younger engineers was marked. To them and to a host of others the name he bore was particularly appropriate, for he was truly a friend.

Mr. Friend is survived by his widow, Josephine (Robinson) Friend, to whom he was married on February 22, 1929; two sons, Paul and Richard; and a brother, Lloyd.

He was a member of the American Railway Engineering Association, the Masonic Lodge, and the Methodist Church.

Mr. Friend was elected a Member of the American Society of Civil Engineers on January 13, 1930.

GEORGE BRADLEY GASCOIGNE, M. Am. Soc. C. E.[1]

Died August 17, 1940

George Bradley Gascoigne was born in Oak Harbor, Ohio, on December 18, 1884, the son of the Rev. Mortimer and Carrie (Bradley) Gascoigne.

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[1] Memoir prepared by Miss A. C. Siegman, Cleveland, Ohio.

He received his preparatory education in the schools of Toledo, Ohio, and was graduated from Ohio State University, Columbus, in 1910, with the degree of Civil Engineer. During his college course, Mr. Gascoigne showed considerable interest in student activities, particularly athletics. Thus he was a member of the baseball team (1906), assistant manager and manager of the football team (1907 and 1908), and athletic editor of the Makio Yearbook (1908). Social qualities and those of leadership were recognized by election to Gamma Phi (athletic association), Varsity O, Triangle (engineering), Bucket and Dipper, Sphinx (honorary), and Phi Delta Theta.

Mr. Gascoigne's first professional work, after graduation, was at Sandusky, Ohio, where he was assistant in the Engineering Department. In 1912, however, he entered the employ of the City of Cleveland, Ohio, participating for ten years in activities that constituted direct preparation for later entry into the field of consultation.

During the years following 1912, the City of Cleveland was facing the necessity of providing for treatment of its sewage. Under the direction of Robert Hoffmann, Hon. M. Am. Soc. C. E., then commissioner of engineering, the young engineer was placed in charge of the recently instituted Subdivision of Sewage Disposal. Concerned initially with the planning of intercepting sewers, the division soon embarked upon sanitary surveys, industrial waste surveys, and, in particular, a sustained program of testing and demonstration methods of sewage treatment. This led, in 1916 and 1917, to classic work with the activated sludge process, and, later, to demonstration with a full-sized mechanical screen. The centrifuging of sludge, sludge drying on covered beds, disinfection, incineration of screenings, and other phases of sewage treatment were subjected to exploratory tests. During this period, also, prominent sanitarians were consulted. The late Harrison P. Eddy,[2] Past-President, Am. Soc. C. E.; the late George W. Fuller,[3] the late R. Winthrop Pratt,[4] the late W. H. Dittoe, and Karl Imhoff, Members, Am. Soc. C. E.; Earle B. Phelps, and other authorities, all contributed in varying degrees to the several undertakings at Cleveland, and with collateral effect upon engineering outlook and reasoning as Mr. Gascoigne was wont to acknowledge with full appreciation.

The investigations and studies by the Subdivision of Sewage Disposal bore fruit in a comprehensive plan of sewage treatment for metropolitan Cleveland. Works for partial treatment, with provision for disinfection and submerged dispersion, were built in the Westerly District. Preliminary works, including the disinfection of rough screened sewage, and submerged outfall, were built in the Easterly District. Supplementing these was a widespread system of interception, some of which involved difficult tunnel construction.

Mr. Gascoigne's career as a consulting engineer began in 1922. He then enlisted the cooperation of William L. Havens and Alfred A. Burger, Members, Am. Soc. C. E., and Frank Woodbury Jones—fellow employees of the city—and established a professional engineering business successively under the names of George B. Gascoigne, and Gascoigne and Associates. This business endured to the day of his death and—significant of his judgment of men—it still exists, under a different name, but otherwise unchanged from the organization that he built up.

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[2] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 104 (1939), p. 1867.

[3] For memoir, *loc. cit.*, Vol. 100 (1935), p. 1653.

[4] For memoir, *loc. cit.*, Vol. LXXXIII (1919–1920), p. 2317.

During his eighteen years' practice as consulting engineer, and exclusively in the field of sanitation, Mr. Gascoigne was responsible for the design and completion of work aggregating a cost of some $50,000,000. In addition, he made studies, reports, and appraisals upon projects of approximately equal amount. Chief among the undertakings were those which brought to completion the essential parts of the Cleveland plan at a cost of about $17,000,000 and inclusive of the large Easterly activated sludge plant. A matter of considerable pride to Mr. Gascoigne was the fact that the entire Cleveland program, covering, in all, some seventy contracts during an interval of ten years, was accomplished without litigation and without claims for "extras" of any substantial amount.

The record of Mr. Gascoigne's professional engagements during the period of his service as consultant includes the following as major works of design and construction:

City	Type of Work
Alliance, Ohio	Outfall sewer and sewage treatment works.
Ashtabula, Ohio	Outfall sewer and sewage treatment works.
Bellevue, Ohio	Gravity supply lines, pumping station, elevated tank, and water filtration plant.
Chattanooga, Tenn.	Sewage treatment works for Brainard Sewer District.
Cleveland, Ohio	Easterly activated sludge sewage treatment project. Southerly sewage treatment works and later extensions. Additions to the Westerly sewage treatment works.
Conneaut, Ohio	Intake crib and breakwater, water filtration plant, and elevated tank.
Delaware, Ohio	Sewage treatment works.
Elyria, Ohio	Outfall sewer and sewage treatment works. Force main and elevated tank. Design of suction well and additional clear well.
Geneva, Ohio	Sewage treatment works. Elevated storage tank and connecting mains.
Jackson, Mich.	Outfall sewers and sewage treatment works (with the Fargo Engineering Company).
Lorain County, Ohio	Water filtration plant and water mains for Avon Lake.
Marion, Ohio	Outfall sewer and sewage treatment works.
Massillon, Ohio	Sewage treatment works.
Medina, Ohio	Outfall sewer and sewage treatment works. Additional pumping equipment.
Muncie, Ind.	Interceptors and sewage treatment works.
New Haven, Conn.	Meadow Street Intercepting Sewer, Boulevard Sewage Disposal Plant, and sludge disposal facilities for the East Street Plant.
Omaha, Nebr.	Sewage treatment works for Papillion Sewerage District.
Oxford, Ohio	Sewage treatment works.
Springfield, Mass.	Intercepting sewers, Indian Orchard and Main sewage treatment plants.

Troy, Ohio	Sewage treatment works.
Vermilion, Ohio	Sewers, and sewage treatment works. Filtration plant improvements. Water works intake.

In addition to these projects, Mr. Gascoigne served as a member of the Board of Review for the sewerage improvements of Duluth, Minn., and also as a member of the Sewage Rate and Water Rate boards for the City of Cleveland. He participated in but little litigation, but he did furnish technical aid to the plaintiff states in the Supreme Court hearings of 1924 on the Great Lakes diversion. He also appeared in the Federal Circuit Court of Wisconsin in Activated Sludge Inc. vs. Milwaukee Sewerage Commission.

In 1928 Mr. Gascoigne spent three months visiting sewage treatment works in England and Germany. Notes of this visit appeared in *Engineering News-Record* for July 19, 1928; an aftermath was a continued though desultory correspondence with Karl Imhoff of Germany and H. T. Calvert and H. C. Whitehead of England.

Although a member of several technical organizations, Mr. Gascoigne wrote but little. "Grit Chambers for Sewage Treatment Works" appeared in the *Transactions* of the Society[5]; "Systems of Sewage Aeration" was presented before the American Society of Municipal Improvements in 1928; and "Chlorination of Sewage and Sewage Effluents" was published in the *Sewage Works Journal.*[6] A number of other and fugitive articles and addresses appeared in the technical press.

In the Society, Mr. Gascoigne's most notable work was done as a member of the Committee on Filtering Materials for Water and Sewage Works from 1928 to 1936. Much of the experimental work on tests of trickling filter mediums was done in his laboratory; it resulted in establishing the present standard sodium sulfate test for this material. Mr. Gascoigne was also a member of the 1937 Committee on Aims and Activities. He was able to give little time to local section affairs, but he supported them generously, even though vicariously, by encouraging and meeting the expense of activity by others of his staff and organization.

Mr. Gascoigne was a member of many technical organizations, including the American Institute of Consulting Engineers, The Institute of Sewage Purification (England), the American Public Works Association (Past-President), the American Water Works Association, the American Public Health Association, Cleveland Society of Professional Engineers, and the Ohio Society of Professional Engineers. He was also a member of the Rotary Club, the Cleveland Chamber of Commerce, the Union Club (Cleveland), Country Club (Cleveland), and the Engineers' Club of New York.

Any appraisal of Mr. Gascoigne as man and engineer must recognize contradictory characteristics. He was essentially dynamic and he gave liberally from his store of nervous energy, but a seeming impulsiveness was usually founded upon painstaking and sometimes prolonged study. He saw to it that his technical staff consisted of diverse types, and he was wont to hold council upon matters of importance—he himself arguing on either side of the question to develop counter opinion. Once the evidence was in, however, action was swift and sure.

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[5] *Transactions*, Am. Soc. C. E., Vol. 91 (1927), p. 496.

[6] *Sewage Works Journal*, 1931, p. 38.

Mr. Gascoigne had social gifts in abundance, founded upon his genuine liking for people. Often he was judged upon these obvious qualities alone and in lack of appreciation of a more solid background. He played hard, but solely in reaction from other periods of hard work in which his concentration was intense. He was an enthusiastic fisherman and golfer.

To epitomize, Mr. Gascoigne's chief quality was that of human understanding—of ability to evaluate properly the viewpoints of client and opponent alike. From this attitude there resulted engineering projects designed with unusual regard for client wishes and needs, although without waiving of technical considerations. As he was prone to say, "The engineering must be sound."

On July 17, 1912, Mr. Gascoigne was married to Myra Sherman of Columbus, who, with two children, George Bradley, Jr., and Ann S. (Mrs. George H. P. Lacey), survives him. He is also survived by his mother, Mrs. Carrie B. Gascoigne.

Mr. Gascoigne was elected an Associate Member of the American Society of Civil Engineers on November 4, 1914, and a Member on July 6, 1920.

LAURENCE ARCHIBALD GOINES, M. Am. Soc. C. E.[1]

DIED DECEMBER 18, 1932

Laurence Archibald Goines, the eldest son of William and Emma (Morgan) Goines, was born in New York, N. Y., on July 23, 1886. His father, a physician, was a native of New York State, and his mother was a Virginian.

Mr. Goines was educated in the public schools of New York City and shortly after graduation moved to Wyoming. During the next five years his work included sub-professional engineering in Western United States, Canada, and Mexico. In 1908 he entered the University of Wyoming, at Laramie, Wyo., and in 1912 received the degree of Bachelor of Science in Civil Engineering. He later took the LaSalle Law Course and studied in various law offices for a period of eight years.

From 1912 to 1919 he was with the Costilla Estates Development Company, San Acacio, Colo., the owner of a 600,000 acre tract of land lying in southern Colorado and northern New Mexico. During the first five years Mr. Goines held responsible engineering positions, both field and office, in the design and supervision of construction of irrigation works, highways, timber production, and land development. During the last two and one-half years with this company he was chief clerk and assistant to the general manager of this and five subsidiary companies, largely in executive work. For long periods he was in full charge of the diverse business of the six corporations.

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[1] Memoir prepared by Robert Fulton, Assoc. M. Am. Soc. C. E.

The Great Western Sugar Company, near Littleton, Colo., employed Mr. Goines as ranch superintendent in building construction and ranch development in August, 1919. Upon completion of this work he entered the consulting engineering field and practiced in Nebraska and Colorado for about a year.

From 1921 to 1925 Mr. Goines was general manager of the South Carolina properties of the C. W. Kress Company. His duties included the reorganization of construction and operating forces; installation of an accounting system; drainage of abandoned rice fields; tide and flood-protection construction; construction and operation of an industrial railroad; and—probably the outstanding feature of his work—the installation of a system of subsurface irrigation following his original design.

In 1925 Mr. Goines came to Alliance, Nebr., as city manager, which position he held until his death on December 18, 1932. His work included the administration of all city departments, including police, fire, streets, engineering, finance, legal, sewer, health, public utilities (muncipally owned electric and water systems), and cemetery. His diverse experiences and education fitted him for this position and, of all his professional work, Mr. Goines most enjoyed his labors in Alliance. The city is not large but, because of its isolated location, it is an important ranching, trading, and railroad center. When Mr. Goines settled there, Alliance was just beginning to emerge from its wild west, wide-open era, and the police department taxed his ingenuity to the utmost. With reform in the air Mr. Goines was called upon to lessen the frequent brawls, cowboy exuberance, and gun fights, and to bring about a proper respect of law and order. He was small in stature but determined in spirit, and the trials of the city police department were studied at home many nights before the required results were obtained.

While in Alliance Mr. Goines served as city engineer during the construction of paving, sewer, and water system extensions aggregating more than $800,000, modernized the electric and water plants, and accomplished notable reductions in the city bonded debt. He also accomplished his greatest ambition—that of making Alliance an outstanding example of successful city management.

Mr. Goines was a member of the Colorado Society of Engineers, the International Association of City Managers, the American Legion, and served as President of the Nebraska League of Municipalities. He held membership in the Blue Lodge, A.F. and A.M., Alamosa, Colorado Consistory, Pueblo, Colo., and the Eastern Star of Alliance, of which he had been elected Patron.

Mr. Goines and Lulu M. Mead were married on July 29, 1910. He is survived by his widow and five children, William Henry, Marguerite Katherine, Laurence Archibald, Patricia Eileen, and Dorothy Jeanne. Two brothers also survive.

Mr. Goines was elected an Associate Member of the American Society of Civil Engineers on November 26, 1923, and a Member on May 12, 1930.

HENRY GOLDMARK, M. Am. Soc. C. E.[1]

Died January 15, 1941

Henry Goldmark left his imprint in technical progress over the entire continent of North America, and in his active professional life of almost sixty years he spanned the periods of empirical and scientific engineering design. In all his work, his aim was to develop new and more accurate methods of design and to accept nothing as correct unless previously proved by either experience or experiment. The variety of work included in his record of accomplishments bears witness to Mr. Goldmark's wide technical knowledge and his rigid adherence to the scientific method.

Henry Goldmark was born in New York, N. Y., on June 15, 1857. His father, Joseph Goldmark, a native of Hungary, and a physician and a liberal member of the Parliament in Vienna, Austria, came to the United States in 1851, fleeing from the conservative reaction following the revolution of 1848. His mother, Regina Wehle, was a native of Prague, Czechoslovakia, a member of a family connected with that city for many generations. She had come to the United States in 1849. One of ten children, Henry Goldmark spent his early life in a then suburban district of the City of Brooklyn, N. Y., adjacent to Columbia Heights, rural enough to have a large garden with cherished grape and fruit trees and yet within sight of the growing metropolis.

His early technical education was at the Collegiate and Polytechnic Institute of Brooklyn, then one of the few American preparatory schools including scientific studies in the curriculum. This was followed by the formal course in arts at Harvard College, in Cambridge, Mass., where he was graduated in 1878 with the degree of Bachelor of Arts. One of his classmates, who remained his closest, life-long friend, was Paul Shorey, the eminent classicist. Young Goldmark showed at Harvard the literary taste which always made him a voracious reader and a student of subjects outside the scientific field. One of the courses which he never forgot and which in his opinion did more than any other to broaden his literary interests was the famous Chaucer course of the beloved Professor "Stubby" Childs.

In accordance with the necessary custom of the day, his technical training was completed by two years' attendance at the Politechnische Schule at Hanover, Germany. Here he formed friendships with English and Dutch students that lasted more than half a century and were kept alive by continuous correspondence among the small group of technicians whose work covered the globe. Returning to America in 1880, Mr. Goldmark started his long service in the advancement of engineering. One of his first important steps in engineering was to affiliate himself with the American Society of Civil Engineers.

The development of both transportation and steel influenced his early work. After a year with the New York, Lake Erie and Western Railroad

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[1] Memoir prepared by Jacob Feld, M. Am. Soc. C. E.

Company and with the Passaic Rolling Mill (Paterson, N. J.) on planning and detailing railroad structures, Mr. Goldmark received an offer to "go West." After a long, tedious journey by train to St. Louis, Mo., and then by coach, he reached the Texas area. There he was placed in charge of the location of the Texas and St. Louis Railroad. Financial difficulties stopped the work just as he was offered a position with the West Shore Railroad Company in 1882.

Then came two years of active design and reconstruction of the bridges for the West Shore Railroad Company, including shop and field inspection. One interesting item that he used to tell of the work during this period concerned an inspection of all concrete masonry culverts, which showed that some were much more resistant to increased traffic and weight of rolling stock than others. A scientific study by Mr. Goldmark led him to the deduction that the additional resistance was due to the fact that, in some of the culverts, discarded worn rails had been embedded in the masonry arches. A very early empirical example of reinforced concrete was thus discovered.

From 1884 to 1891, Mr. Goldmark took an active part in the development of bridge design and construction for several railroad lines, being one of the first to utilize steel for bridge manufacture. After inspection work on steel at the Johnstown, Pa., shops for the Henderson Bridge and for the St. Louis-San Francisco Railway Company, he was called to examine and recommend methods for strengthening all existing bridges of the Southern Kansas Railroad, the Boston and Maine Railroad, the Kansas City, Fort Scott and Memphis Railway, Chesapeake and Ohio Railway, and the Memphis and Birmingham Railroad companies. This was the period of increasing train loadings, and his technical skill was useful in saving existing investments in the railroad bridge structures. Near the end of this period, the subject of dams also attracted his attention, since water conservation was just as important as water crossings for the pioneer western railroads. In 1891 he contributed to the "Report of the Committee on the Cause of the Failure of the South Fork Dam."[2]

With the staff for designing the Columbian Exposition for Chicago, Ill., Mr. Goldmark contributed structural designs for such buildings as the Machinery Hall from 1891 to 1893. The next four years were spent in a great variety of structural and consulting problems; the most noteworthy were the design of a steel dam for the Ogden (Utah) Pioneer Electric Power Company, in 1896, and the design of a concrete viaduct over Rock Creek Park, Washington, D. C., in 1897. In this well-known bridge, known as the Connecticut Avenue viaduct, Mr. Goldmark always took special pride. The wide range of his work explains the clear insight in the discussions contributed by Mr. Goldmark to Society papers from 1892 to 1897 on subjects covering the continuous bridge at Memphis, Tenn., bridge painting, cost of sewer construction, distortion of riveted pipe by backfilling, pressures resulting from changes of velocity in pipes, portland cement concrete, and structural steel specifications. In 1898 he was awarded the Society's Thomas Fitch Rowland Prize for his paper on "The Power Plant, Pipe Line and Dam of the Pioneer Electric Company at Ogden, Utah."[8] In this paper, he described how economic studies, taking into account the availability of materials in the locality, had to be taken into consideration in the choice of alternate designs.

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[2] *Transactions*, Am. Soc. C. E., Vol. XXIV, January–June, 1891, p. 467.

In 1897 Mr. Goldmark accepted an appointment as designing engineer with the U. S. Board of Engineers on Deep Waterways, in completing the study of a U. S. Ship Canal from the Great Lakes to salt water. He left this work in 1899 when his former railroad friends insisted that he must not spend his talents on water transportation; so he became resident engineer of the combination railroad and highway bridge over the Missouri River at Atchison, Kans. His contributions to the history of locks and lock gates for ship canals formed the subject of a series of lectures in 1899, reprinted by Cornell University, at Ithaca, N. Y.; the Detroit Engineering Society; and others.

Upon the completion of this work, the Canadian Pacific Railway Company, which had drawn heavily on United States railroads for its executive talent, called upon Mr. Goldmark to go to Montreal, Quebec. There, from 1902 to 1906, he designed and supervised construction of the locomotive and car shops, and the power houses at Montreal (Quebec), Winnipeg (Manitoba), and Fort William (Ontario), a program involving the expenditure of four million dollars. In 1906, together with his younger brother, Charles, an electrical and mechanical engineer, he started the design of the locomotive works for the Canadian Locomotive Company at Kingston, Ontario. Soon, however, another call forced him to turn this project over entirely to his brother.

With such a variety of technical contributions to the engineering development over a period of twenty-five years, and especially with his deep knowledge of the subject of locks and lock gates as demonstrated in the work for the Deep Waterways Board, it is small wonder that the opportunity came to him in 1906 to design the locks for the Panama Canal. This great work occupied eight years in the prime of his professional life; it gave ample scope for his unusual talents in planning major features and in designing their minor details. It was at least the most spectacular, if not the most important, of his great engineering accomplishments. The first two years on this assignment as designing engineer for the Isthmian Canal Commission were spent in Washington, D. C., on designs and details; then followed six years when he was stationed at Culebra in the Canal Zone.

Personally Mr. Goldmark is credited with the design of the lock gates, valves, electrical equipment, and the emergency dams of all the Panama locks. In all, he designed and superintended the erection and installation of ninety-four mitering lock gate leaves and twenty-four chain fenders, as well as the movable caisson emergency dams. At the completion of this monumental work, he was instrumental in having issued to each member of his staff a complete record of the designs in the form of a bound volume of all contract drawings. As ever, he was thoughtful of the value of broadcasting the new information that he had developed.

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[8] *Transactions*, Am. Soc. C. E., Vol. XXXVIII, December, 1897, p. 246.

The Panama work was the incentive for a series of papers on various phases of the work.[4, 5, 6, 7] In addition he gave innumerable lectures on these subjects before engineering societies and at many engineering schools. His simple method of presentation, scholarly bearing, and descriptive explanation made these lectures very popular. Only when his health became impaired did he refuse invitations to present these lectures before student bodies.

Upon his return from Panama, in 1914, Mr. Goldmark entered the consulting field, with headquarters in New York City. He remained active until 1928. Quite naturally, his greatest interest was in canal work. Associated with the late Gen. George W. Goethals,[8] M. Am. Soc. C. E., from 1914 to 1917, he undertook the design of the New Orleans (La.) Inner Navigation Canal, including the lock gates, electrical equipment, and a new type of movable steel stop-log emergency dam. For designing the locks, data on soil pressure and soil bearing were obtained by a careful set of tests. The resulting design of a floating structure has proved successful as well as economical. He published several reports of the novel features in these designs, including a paper contributed to the Society.[9] At the completion of the New Orleans Canal, Mr. Goldmark became a member of the Board of Consulting Engineers on the New York Barge Canal.

In 1919 the Japanese Government commissioned Mr. Goldmark to prepare designs for a high tidal gate for the Chemulpko Harbor in Korea. During this last decade of his active practice, a variety of interesting problems was undertaken. As chairman of the committee to pass on the designs for the U. S. Navy dirigible airship "Shenandoah" and for the U. S. Army semi-rigid airship "RS-1," he delved into the subject of aerodynamics with the same vigor that he showed in his earlier study of electrically operated lock gates.

From 1923 to 1926, the Society honored Mr. Goldmark by appointment as one of its three representatives to the National Research Council, an honor which he prized most highly. Associated with the engineering firm of Murray and Flood, he investigated the hydraulics of the proposed Santee Cooper hydroelectric project in South Carolina and the economic possibility of the proposed tidal-electric dream in Passamaquoddy Bay, Maine. In the latter, Mr. Goldmark's report was definite that no fixed power could be expected without the construction of auxiliary steam plants or high reservoir storage.

In 1926 he was engaged by the International Telegraph and Telephone Company to review plans prepared for the construction of seven radio towers,

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[4] "Lock Gates, Chain Fenders and Lock Entrance Caisson," presented before the International Eng. Cong., San Francisco, Calif., 1915, and printed in *Transactions*, International Eng. Cong., Vol. II.

[5] "The Distribution of Stresses in Mitering Lock-Gates, with Special Reference to the Gates on the Panama Canal," by Henry Goldmark, *Transactions*, Am. Soc. C. E., Vol. LXXXI, December, 1917, p. 1621.

[6] "Tests of Chain Fenders in the Locks of the Panama Canal," presented before the Canadian Soc. of Civ. Engrs., February 25, 1918.

[7] "Manufacture and Testing of Large Chains for Fenders in the Panama Canal Locks," presented before the Western Soc. of Engrs., 1919, and printed in *Journal*, Western Soc. of Engs., February, 1919, Vol. XXIV, p. 2.

[8] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1813.

[9] "Emergency Dam on Inner Navigation Canal at New Orleans, La.," *loc. cit.*, Vol. 92 (1928), p. 1589.

each 1,000 ft high, for a proposed central transmitting station at Shanghai, China. The report on this project was so complete that his office was given the design problem of analyzing the stresses in antenna mats to be hung from these towers, spaced 2,600 ft apart in square arrays. The solution of this space problem in cable design was one of Mr. Goldmark's prize reports.

Immediately after this unusual problem came an opportunity to design the loading equipment for the Seatrain ships to carry fully loaded railroad cars between Habana, Cuba, and ports in the United States. The success of these designs as applied by the Seatrain Corporation was a fitting conclusion to this period of consulting practice.

Having passed the age of three score years and ten, Mr. Goldmark retired from active practice to the quiet of private life at Nyack, N. Y. Part of his time he spent with his sisters in their home at Hartsdale, N. Y. He spent his time in reading, walking, and some physical activity, and made many close friends there. Once, in 1935, he answered a call to inspect the designs for the Grand Coulee Dam in the State of Washington. The strenuous trip and outdoor exposure in bad weather proved too much for him, and he was rushed back to New York after a siege of pneumonia. His rugged constitution won the battle, and, with the exception of some difficulty in orienting the dates of his memories, he seemed to have regained his health within a few months.

Another effect was that he had to forego his regular bus trips to New York to attend meetings of the Society and of the Harvard Engineering Society. To the end, his interest in new phases of engineering knowledge was most marked. Only a few months before his death, he was beginning a systematic study of recent soil mechanics papers. Many were his friends, old and young, in the profession. He valued their esteem and the sentiment was reciprocal. So it was that he kept in touch with them for many years after active association. To them his death, resulting from an automobile accident, was a great shock.

Always courteous, Mr. Goldmark was rather mild in his manner. He was exact and courtly in speech, abstemious in habit. A certain sweetness of character and bearing marked him. He was a credit to the highest traditions of the profession he so loved and honored.

Among other connections, Mr. Goldmark was a member of Phi Beta Kappa, the Institution of Civil Engineers of London, England, the Engineers Institute of Canada, the Western Society of Engineers, the Harvard Club of New York, and the Engineers' Club of New York. He was a member of the Committee on Lock Valves of the Waterways Division of the Society and contributed to Manual No. 3 on "Lock Valves," issued in 1930. Just before his retirement he prepared a complete work (in collaboration with the writer of this memoir) on the subject of fixed and movable steel dams[10] for the American Institute of Steel Construction.

On September 25, 1892, he was married to Louise Condit Owens in Kansas City, Mo. During their stay in Utah, she became seriously ill, which forced them to go East. She died on July 18, 1897. On June 8, 1899, he was married to Mary Carter Tomkins, in Detroit, Mich. Two children of this marriage, Elliott Regina (born 1900) and Henry, Jr. (born 1904), survive him. To these children Henry Goldmark was greatly devoted. Also surviving him are two brothers, James Goldmark and Charles J. Goldmark, and five sisters, Alice (Mrs. Louis D. Brandeis), wife of the former Associate Justice of the United States Supreme Court; Helen (Mrs. Felix Adler); and the Misses Susan, Pauline and Josephine Goldmark.

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[10] "Steel Dams and Gates," by Henry Goldmark and Jacob Feld, 1930 (not published).

Mr. Goldmark was elected a Junior of the American Society of Civil Engineers on May 7, 1884, and a Member on June 6, 1888.

THOMAS TRIPLETT HUNTER HARWOOD, M. Am. Soc. C. E.[1]

Died November 21, 1940

Thomas Triplett Hunter Harwood was born at West Point, N. Y., on April 20, 1864, the son of Col. Franklin Harwood, U. S. Army, and Julia (Hunter) Harwood. As his father was transferred to various stations about the country, the son obtained his schooling at Buffalo, N. Y., Baltimore, Md., and Detroit, Mich. He attended the Rensselaer Polytechnic Institute, at Troy, N. Y., but the sudden death of his father made it necessary for him to leave the Institute. He accepted a position as assistant inspector of dredging on building the jetty for the Newburyport (Mass.) Harbor. He continued with the United States Engineer Department for thirty-five and one-half years.

Mr. Harwood assisted in making the first survey for the Sandy Bay Breakwater at Rockport, Mass., and directed the placing of the first stone of the Breakwater in 1885. He also drew the specifications for this important work, and was inspector on the project until 1887, when he returned to the Boston (Mass.) office.

He was promoted in 1888 to assistant engineer, which was as high a position as he could attain in the Engineer Department, as it was necessary to be an Army officer and a graduate of the U. S. Military Academy, at West Point, to obtain the position of engineer. He was assigned, and had charge of, river and harbor work from the mouth of the Merrimack River to Cape Cod, Massachusetts, and also a considerable amount of work on Lake Champlain, New York. He was in charge of fortification work in Boston Harbor during the Spanish-American War.

In 1889 Mr. Harwood was married to Clara R. Grimes. He then moved to Rockport, where he was a resident until his death. In addition to his widow, he is survived by two brothers, Franklin B. and Julian H.; a sister, Virginia (Mrs. Thomas G. Cantlett); and several nephews and nieces.

He was a Past Master of Ashler Lodge of Masons, a member of the Gloucester (Mass.) Rotary Club, a director of the Gloucester Safe Deposit and

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[1] Memoir prepared by Louis A. Rogers, Treasurer, Rockport Paving Block Corporation, Rockport, Mass.

Trust Company, and vice-president and director of the Gloucester Chamber of Commerce. For ten years he was chairman of the Special Water Committee appointed to improve the quality and increase the supply of water furnished to the Town of Rockport, and, in many instances, acted in an advisory capacity for construction of sea walls and other improvements in the town.

Mr. Harwood was elected a Member of the American Society of Civil Engineers on April 5, 1910.

JAMES HENRY HAYLOW, M. Am. Soc. C. E.[1]

Died August 28, 1940

James Henry Haylow was born on June 18, 1870, at Woodstock, Ontario, Canada. He was the son of James Haylow, a native of Ireland, and Lydia (Sears) Haylow, a native of Canada.

James Henry Haylow was educated at the Woodstock Collegiate Institute, and the Toronto School of Practical Science, at Toronto, Canada, where he pursued a scientific and engineering course. He was graduated in 1891 with the degree of Bachelor of Science.

Following his graduation, Mr. Haylow accepted a position in the engineering department of the New York Central and Hudson River Railroad at Buffalo, N. Y. Two years later he was appointed assistant engineer by the Board of Park Commissioners of Cleveland, Ohio. In this position he had charge of drainage, and street and bridge construction for five years.

His next engagement was with the Lorain Steel Company of Johnstown, Pa., for two years, constructing street-car lines and steam railroads. Subsequently, he became principal assistant engineer for Ford, Bacon and Davis, Engineers of New York, N. Y., representing them on street railway construction at Edgewater, N. J., Birmingham, Ala., and Nashville, Tenn.

He then spent a year with Jones and Laughlin Steel Company of Pittsburgh, Pa., on heavy foundation construction, followed by a year on railroad location with the Trans-Continental Railway of Canada.

In 1906, Mr. Haylow went to Memphis, Tenn., as chief engineer and superintendent of maintenance of way for the Memphis Street Railway Company, which position he occupied until his retirement in 1938. During that time, the company's entire system was rebuilt along modern lines and in accordance with current practise.

Mr. Haylow was a patient, energetic worker, and a man of sterling character combined with a jovial spirit that endeared him to all his acquaintances.

He was a charter member of the Engineers Club of Memphis, and was its President in 1925. He was active at all times in engineering society affairs. He was a member of a group of engineers, meeting for the purpose of organizing the Mid-South Section of the Society, aboard the *S. S. Norman* when it capsized and sank in the Mississippi River about twenty miles below Memphis, on May 8, 1925, with a loss of twenty-three lives.

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[1] Memoir prepared by L. L. Hidinger, M. Am. Soc. C. E.

In 1903, Mr. Haylow was married to Helen Rose of Woodstock, Ontario. To this union were born two daughters, Helen Rose and Eugenia Elizabeth (Mrs. Reid McGregor), who survive him. In 1926, he was married to Alta Mae Pence of Memphis, who also survives him.

He was a member of the Odd Fellows and of the First Unitarian Church.

Mr. Haylow was elected an Associate Member of the American Society of Civil Engineers on December 5, 1906, and a Member on October 3, 1911.

CARL SWING HERITAGE, M. Am. Soc. C. E.[1]

DIED AUGUST 31, 1940

Carl Swing Heritage, the son of Charles Davis and Mary Emma (Swing) Heritage, was born at Pennsville, Salem County, N. J., on September 3, 1880. His early education was received in preparatory school at Temple University, in Philadelphia, Pa. In June, 1904, he was graduated from the civil engineering course at Lehigh University, in Bethlehem, Pa.

While still in college, he worked one summer (June to September, 1903) as an assistant on the Engineer Corps of the Chicago Division of the Baltimore and Ohio Railroad Company. This work consisted of surveying and drafting. After graduation he entered the employment of the Pennsylvania Steel Company, in the bridge and construction department, in the office of the engineer of erection at Steelton, Pa., and remained in the service of that company until September, 1904. Then he entered the service of the McClintic Marshall Company at Pittsburgh, Pa., where he did estimating and designing in the drafting room of the chief engineer's office. He also served as resident engineer on erection of steel buildings and as assistant to the manager of erection for the McClintic Marshall Company until August, 1909.

In September, 1909, he entered the employment of the engineering firm of Waddell and Harrington, Consulting Engineers, at Kansas City, Mo., and continued in their service as a bridge designer until October, 1910, when he was employed by the Kansas City Southern Railway Company, at Kansas City, as a bridge draftsman. In 1915 he was appointed bridge engineer for the Kansas City Southern Railway Company, which position he held until his death.

An engineer of ability, he combined with his technical knowledge a winning personality. His loyalty to his employers was outstanding, and his contacts gained him many strong friends who respected him for his integrity, his conscientious work, his uprightness of character, and his sympathetic feeling. His intimate friends will not forget his kindness, courteous consideration, and benevolence, which endeared him to all with whom he came in contact.

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[1] Memoir prepared by A. N. Reece, M. Am. Soc. C. E.

Mr. Heritage was never married. He devoted his spare time to reading and association with his friends. He was a member of the American Railway Engineering Association and served on many committees of that organization. He was also a member of the American Railway Bridge and Building Association, of which he was a Past-President; the Phi Gamma Delta Fraternity; the Engineers Club of Kansas City; Ivanhoe Country Club; and the following Masonic bodies: Blue Lodge, Royal Arch Masons, Knights Templar, and Shrine.

Mr. Heritage was elected a Member of the American Society of Civil Engineers on October 1, 1926.

FRANK DUDLEY HOLBROOK, M. Am. Soc. C. E.[1]

DIED MARCH 31, 1941

Frank Dudley Holbrook was born in Boston, Mass., on September 1, 1872. He was the son of Samuel and Carrie (Smith) Holbrook of English ancestry.

His technical education was obtained at Norwich University, at Northfield, Vt., from which he was graduated in 1893 with the degree of Civil Engineer.

His engineering experience began immediately following his graduation, and his more important professional engagements were as follows:

From June, 1893, to December, 1897, he served as rodman, instrumentman, and inspector, and later as superintendent of construction on sewers and on grade crossing elimination in Brockton, Boston, and Natick, Mass., with a brief employment on railway location in Mexico.

From March, 1898, to September, 1900, he was employed as superintendent of construction and inspector for the U. S. Government Service on coast fortifications in the eastern entrance of Long Island Sound.

From September, 1900, to November, 1915, Mr. Holbrook was employed as junior engineer and later as U. S. assistant engineer in responsible charge of construction of various locks and dams on the Ohio River, including particularly Lock and Dam No. 15, and with special service on Lock and Dam No. 11, No. 14, No. 16, No. 18, and No. 19. During this period he was Chairman of the Board that developed a plan for operation of locks and dams by water power.

From March, 1916 until 1922, Mr. Holbrook was in private engineering practice at Decatur, Ill., as a member of the engineering firm of Miller-Holbrook Engineering Company (later Miller-Holbrook-Warren and Company, then Holbrook, Warren and Andrews, and still later Warren and Van Praag, Inc.). He specialized in the design and supervision of construction of a wide variety of municipal improvements, and also in the design and supervision of reinforced concrete country grain elevators in the central grain belt of Illinois. He was president of the engineering firm which bore his name.

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[1] Memoir prepared by Willis D. P. Warren, M. Am. Soc. C. E.

During the period from 1922 to 1933, Mr. Holbrook was employed as engineer for the Sanitary District of Decatur, and also for the Park District of Decatur, dividing his time between the two organizations. He was in responsible charge of important sewage treatment plant improvements for the Sanitary District, and in charge of park improvements involving construction of pavements, park pavilions, sewers, and other similar improvements.

Mr. Holbrook's professional life was spent on a wide range of engineering projects. With his brilliant mind, sound technical training, and rugged honesty, he proved himself an engineer of exceptional ability. The most important of the many engineering works under his direction were the locks and dams on the Ohio River, the reinforced concrete grain elevators in the rich grain belt of Illinois, and the sewage treatment works at Decatur.

Mr. Holbrook was a man of dignified and yet friendly disposition, of gentlemanly instincts, and of high moral character.

Aside from his busy life as an engineer, he had numerous other interests and accomplishments. He had a wide knowledge and appreciation of classical music. In his earlier years he played the cornet and was also considered an excellent singer.

He always loved flowers, and upon his retirement from active engineering practice in 1933 he devoted much time to his garden. In recent years his home in Decatur was considered a "Garden Spot," and his flowers, shrubs, trees, lawn, and house created a landscape effect which was truly pleasing and artistic.

Mr. Holbrook was a member of the Decatur Club and for years played bridge with a circle of intimate friends. In recent years he played golf, and with his natural love for the outdoors, he found great diversion and pleasure in the game.

Mr. Holbrook was an active and enthusiastic Rotarian, and, in company with his wife and his son, Frank Dudley, Jr., he attended Rotary International on June 6 to 12, 1937, at Nice, France. The Holbrook family sailed on the Italian Liner *Roma,* stopping at Maderia, various points in Africa, Gibraltar, and Naples, Italy. Also, in company with his family, he visited Mexico in 1939. His travels included a trip to Canada and many motor trips to various parts of the United States.

He was proud of his membership in Ohio's oldest Masonic Lodge, No. 1, A. F. and A. M. of Marietta, Ohio.

Following his retirement from the engineering profession he made a broad and scientific study of investments and his judgment and success in this particular field stamped him as a man of exceptional financial ability.

He was married to Carrie L. Chapman on June 30, 1900, in Westerly, R. I. He is survived by his widow, a daughter, Martha (Mrs. J. T. Maxwell), and a son, Frank Dudley Holbrook, Jr.

On December 5, 1939, he was awarded a Life Membership Certificate by the American Society of Civil Engineers at the annual meeting of the Central

Illinois Section of the Society held in the Elks Club, Springfield, Ill. Mr. Holbrook had been a member of the Society for a period of thirty-five years.

Mr. Holbrook was elected an Associate Member of the American Society of Civil Engineers on February 1, 1905, and a Member on January 7, 1908.

HUGH BRAXTON HOOPER, M. Am. Soc. C. E.[1]

DIED AUGUST 29, 1939

The death of Hugh Braxton Hooper on Tuesday, August 29, 1939, was indeed a great loss to the Nashville District of the United States Engineer Department.

Mr. Hooper was born in Nashville, Tenn., on June 13, 1888, the son of Thomas Benton and Lura Frances (Adams) Hooper. His early education was obtained at Fogg High School in Nashville, and his great store of engineering knowledge was obtained by continued reading, research, and study, and by detailed experience obtained on many types of engineering work. Through this constant application he attained prominence in the U. S. Engineer Department.

Mr. Hooper's long and meritorious service in the government engineering field began on August 30, 1909, one day less than thirty years before his death. He had an exceptionally good record with the Department, serving in the Chattanooga, Florence, and Nashville districts, in various grades including that of senior engineer, to which he was promoted on November 1, 1938. From 1922 to 1930 Mr. Hooper was in charge of the extensive and varied field work in connection with the survey of the Tennessee River and its tributaries for the projects for the combined interests of navigation, power, and flood control, and, in particular, the investigation of the Cove Creek (Norris) Dam on the Clinch River. Prior to his death he was in charge of the Tennessee River Operations, Navigation, and Construction Section of the Nashville District. He had great executive ability in planning and directing engineering work.

Before the World War, Mr. Hooper was a member of the First Tennessee Infantry, and saw service on the Mexican Border and at Reelfoot Lake during the Nightrider Uprising. During the World War years he was a major in the field artillery, and made a very distinguished military record in command of the 2d Battalion, 115th Field Artillery, under Col. Harry S. Berry. This battalion was composed mainly of volunteers from Nashville, Springfield, Columbia, and other middle Tennessee towns. After the Armistice Major Hooper was transferred to the Army of Occupation and stationed at Polch, Germany. Shortly after his return to the United States from overseas duty, and subsequent to his discharge, he was commissioned a Lieutenant Colonel, Field Artillery Reserve.

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[1] Memoir prepared by S. A. Weakley, M. Am. Soc. C. E.

Colonel Hooper was held in high esteem in both engineering and military circles, and was a member of the Society of American Military Engineers and many other military and civil organizations. Engineering was his work and chief interest, but his hobbies included reading, nature study, and stamp collecting. His outspoken honesty and integrity made him respected and deservedly popular with those with whom he came in contact. His passing was most untimely and he will be keenly missed.

Colonel Hooper was elected a Member of the American Society of Civil Engineers on October 30, 1933.

MALVERD ABIJAH HOWE, M. Am. Soc. C. E.[1]

DIED MARCH 15, 1941

Malverd Abijah Howe, the son of Dr. Asa and Lucy Ann Frances (Cummings) Howe, was born on December 9, 1863, in Northfield, Vt. He was graduated from Northfield High School in the Class of 1879, and received the degree of bachelor of Science from Norwich University, in Northfield, in 1882. Later, he received the degree of Master of Science from the same university. In 1886 he was graduated from the Thayer School of Civil Engineering, at Dartmouth College, in Hanover, N. H.

After his graduation from Dartmouth, he worked for a few months with the Edge Moor Bridge and Iron Company, in Wilmington, Del. Then he accepted a position as instructor of surveying and drawing at Harvard University, in Cambridge, Mass. In 1887 he was appointed professor of civil engineering at the Rose Polytechnic Institute, in Terre Haute, Ind., and did much to build up the Civil Engineering Department. During this time he kept up an outside practice of his profession, confined principally to buildings and bridges. He frequently did consulting work, and designed and supervised the erection of the Wabash River Bridge in Terre Haute. He was superintendent on several public buildings and private works in Indiana, and the Pennsylvania Railroad Company employed him to recalculate the stresses in the Eads Bridge, in St. Louis, Mo.

Mr. Howe was the author of many books, and wrote numerous articles for engineering journals and society transactions, including the "Theory of the Continuous Girder," "Tables for Use in Application of Methods of Least Squares," "Retaining Walls for Earth," "A Treatise on Arches," "Design of Simple Roof Trusses in Wood and Steel," "Symmetrical Masonry Arches," and "Foundations and Masonry." He retired from the teaching profession in 1916 and returned to Northfield where he devoted his time to study and to the revision of his engineering textbooks.

In addition to life membership in the Society, Mr. Howe was a member of the Permanent International Society, the Association Navigation Congresses, the Society for the Promotion of Engineering Education, and the American Society for Testing Materials.

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[1] Memoir prepared by A. E. Winslow, Assoc. M. Am. Soc. C. E.

In 1897 he declined the offer of a position on the engineering staff of the Imperial University of China; he also declined an offer from the British Government during World War I to serve in an engineering advisory capacity.

He was married on June 25, 1888, to Jessie White of Emporia, Kans., who, with one son, Homer Asa Howe, M. Am. Soc. C. E., survives him.

Mr. Howe was elected an Affiliate of the American Society of Civil Engineers on May 7, 1890, and a Member on January 1, 1896.

PERCIVAL MAXIMILIAN INFANTE, M. Am. Soc. C. E.[1]

DIED OCTOBER 9, 1940

Percival Maximilian Infante was the son of Charles and Almarinda Infante and was born in London (Highgate), England, on September 28, 1880. He served as a premium apprentice in the shops of the Barrowfield Iron Works, Glasgow, Scotland, and studied at the West of Scotland Technical College (later the Royal Technical College of Glasgow). He was a student under Lord Kelvin at the University of Glasgow, from which he was graduated in 1902 with the degree of Civil Engineer.

After graduation he worked in Yorkshire, England, for R. Dempster and Sons, Ltd., of Ellands, and Drakes, Ltd., of Halifax, as a draftsman; and then in London successively for Gwynne and Company and for Edward Wood and Company as designer. He became inspecting engineer for the Sturtevant Engineering Company, of London, in 1905, on heating and ventilating stations and factories all over Europe.

In 1907 he undertook an engagement with Sir Douglas Fox and Partners, Engineers, of London, which developed into an association of some ten years and covered work in Portuguese West Africa (Angola) and the Republic of Colombia, in South America. A gifted linguist, speaking French, Spanish, Portuguese, and Italian with fluency and precision, he was able to acquire a working knowledge of the African native dialects and to carry on effective work with an organization which included Portuguese convict labor, native tribesmen, and a variety of others, as well as the regular construction forces.

At first as maintenance engineer and surveyor and then as assistant chief engineer, his resourcefulness and ability in overcoming the obstacles to railroad location through arid mountainous territory having but one village (a native kraal) in a length of 360 miles resulted in his early appointment as chief engineer. As such, he built the Benguela Railroad from Lobito Bay, on the West Coast, into the interior, the object being to provide an outlet to the

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[1] Memoir prepared by J. H. Granbery, M. Am. Soc. C. E.

Atlantic Ocean from the Lake Tanganyika country and the southern part of the Belgian Congo.

Some idea of the difficulties encountered in this construction may be obtained from the fact that a railroad haul of fifty miles was required for all water used on a large part of the work. A rack road was necessary to overcome seven miles of steep grades in one section, and in this region, virtually waterless under ordinary conditions, such rainfall as occurred was torrential and destructive.

The material brought by these sudden floods had a way of blocking the culverts through the accumulation of debris at the upper end, which had to be cleared away afterward to prevent washouts or slides. This was not only troublesome but dangerous, as the men who did the work were likely to be confronted by an angry lion that had chosen the blocked culvert as a secluded retreat and resented being disturbed.

What became known as " Infante's two foot openings " were substituted for the culverts and proved so effective that they were used throughout that work, and later, in the South American work as well. These openings were vertical spaces, extending from the dry bed of the drain to the base of rail, between walls or piers carrying a sleeper or railroad tie at the top of each pier. The lions would not use these spaces, the 2-ft width evidently being too restricted for their comfortable repose, and lacking the privacy afforded by a dark culvert. These openings permitted more rapid discharge than did the culverts and they were not blocked as completely, nor as often.

In conjunction with the railroad location and construction (which included bridges and structures), Mr. Infante was called upon to do extensive prospecting for water and to design a hydroelectric plant taking water from an intake on the Katumbela River twenty-five miles from the plant. He built the steel pier at Lobito and other port works, dams, and ropeway railways before his work in Africa was finished, after which he continued the firm's work in Colombia, South America.

Here, as chief engineer of the Cartagena Railway Company, he was charged with railroad construction from Cartagena to Calamar and with the reconstruction of reinforced concrete and timber piers in Cartagena Bay. For Ropeways, Ltd., of London he constructed sixty miles of ropeway between Mariquita and Manizales; and for the Jenks' interests in London he prospected, designed, and erected the Cartagena Waterworks, the Girardot hydroelectric plant, and the Cali plant on the Cauca River.

As consultant to the Colombian government he made several railroad studies, hydrographic studies of the ports of Cartagena (where he made extensive soundings and designed a new pier), Calamar, and Buenaventura and was commissioned by that government to make hydrographic studies of the Magdalena River for possible canalization and to make surveys for a railroad from Bogota to the Atlantic Coast.

In private practice he did building construction work over the entire republic; he installed the drainage system and paved streets for the City of Cartagena; and for the City of Fúqueme he bored 2½ miles of tunnel near

Chiquinquirá for drainage and reclamation works around the estuary of the Ubate River.

He converted two yachts (one formerly belonging to the Sultan of Morocco) into gunboats for the Colombian Navy. One of these, the Cartagena, later reconverted to a freighter and renamed the Bolivar, was in New York harbor waters at the time of his death.

Mr. Infante came to the United States in 1917 and was designer and later superintendent of heavy concrete construction for Westinghouse Church Kerr and Company at Air Nitrates Plant No. 2 at Muscle Shoals, Ala., and as such was responsible for a remarkable piece of work—a retaining wall 150 ft high and 7 ft thick, several hundred feet long, built along the river to prevent caving of the bluff.

He was afterward employed by the Bethlehem Steel Corporation at Sparrows Point, Md., by the Consolidated Gas Company in New York, N. Y., and then by Gibbs and Hill on electrification of the Virginian and Norfolk and Western railroads and on yard improvements. After this he served as assistant engineer of design on the New York Central's West Side Improvement and Elimination of Grade Crossings.

From 1926 to 1929 he was associated with Waddell and Hardesty, in New York City, designing bridges, and during this period was concerned with work on the Goethals and Kill van Kull bridges. He then opened an office as consulting engineer in New York City which he maintained until late in 1938, though the general stagnation of business soon confined his efforts to such private practice as was still at his command.

In the meantime, from November, 1932, through the greater part of 1935, he served with the U. S. Coast and Geodetic Survey on isostatic and geographic computations. A lively interest in related fields had already prompted his authorship of "An Introduction to the Study of Terrestrial Magnetism."

In October, 1938, he became assistant engineer designer in the Board of Water Supply of the City of New York, the position in which he was employed at the time of his death.

In 1913 Mr. Infante was married to Maud Violet McConkey Hamilton of Hastings, England, daughter of Capt. Arthur Hamilton, of the Royal Artillery, and Maud Ellen Pike. He is survived by his widow and a daughter, Yolanda Elsie.

Mr. Infante was a man of good breeding, courtesy, and kindness; distinguished for a fine sense of honor, strict regard for his obligations, and consideration for the rights and feelings of others. He was quick to recognize and give credit where it was due; scornful of incompetence masquerading as commanding ability; his indignation with any form of injustice was shown by his purchase of an African native to save the latter from a murderous beating at the hands of his brutal German master. Mr. Infante freed the native and gave him a job.

Mr. Infante was proud of his profession with a keen sense of its dignity, but with the modesty of a student—for he was always a student. He was averse to any form of publicity for himself; the importance of the works he accomplished and the extent of his professional attainments can be judged by the statements of others who were conversant with them, or by the works themselves. One would never learn it from him. Perhaps that is his best memorial.

Mr. Infante was elected a Member of the American Society of Civil Engineers on November 12, 1928.

WILLIAM ATLEE JAMES, M. Am. Soc. C. E.[1]

Died July 3, 1940

William Atlee James, who retired from the position of assistant chief engineer of the Western Lines, Canadian Pacific Railway Company, at Winnipeg, Manitoba, on December 31, 1927, died at Lydiatt, Manitoba, on July 3, 1940. He was born at Ebensburg, Pa., on June 4, 1864, the son of Thomas I. and Mary (Evans) James—both born in Pennsylvania. He entered railway service in September, 1883, as flagman on the Burlington and Missouri Railroad in Nebraska.

After serving in other positions with that company, and with the Great Northern and the Butte, Anaconda and Pacific railway companies, he began work for the Canadian Pacific Railway Company in June, 1898, as assistant engineer on construction. He served as transitman for a while, and then he was appointed to the following positions: March, 1901, assistant engineer, Lardo Wharf, British Columbia; May, 1901, locating engineer, Abbotsford, British Columbia; July, 1901, locating engineer, Lardo; August, 1901, resident engineer, Field, British Columbia; May, 1902, locating engineer, Ontario and Quebec divisions; September, 1902, locating engineer, Field; July 1, 1903, assistant engineer, Broadview, Saskatchewan; December 1, 1904, engineer, Western Lines; August 21, 1905, division engineer, Western Lines east of Winnipeg; January 1, 1907, division engineer, Kenora, Ontario; June 1, 1910, division engineer, Western Lines; January, 1913, engineer of construction, Western Lines; and July 1, 1918, assistant chief engineer, Western Lines. Between 1900 and 1904, he was connected prominently with the survey of most of the new Canadian Pacific lines and was in charge of the construction of the 5-mile Connaught Tunnel in the Selkirk Mountains and the double-tracking of the Winnipeg-Fort William line.

He is survived by his widow and three sons, William Atlee James, Jr., F. R. James, and G. James.

Mr. James was a life member of the Engineering Institute of Canada, and a member of the American Railway Engineering Association.

Mr. James was elected a Member of the American Society of Civil Engineers on July 1, 1909.

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[1] Memoir prepared by F. W. Alexander, Asst. Chf. Engr., Canadian Pacific Railway Company, Winnipeg, Man., Canada.

SIR HENRY JAPP, M. Am. Soc. C. E.[1]

DIED APRIL 8, 1939

Sir Henry Japp, K.B.E., was born on the east coast of Scotland, in the Town of Montrose, on June 6, 1869, the son of John Japp and Isabella (Middleton) Japp.

His early tuition was given him at the Academy of his home town and his technical education was received at University College, Dundee, Scotland, under Prof. James Alfred Ewing and at Finsbury Technical Institute, in London. These technical studies were conducted concurrently with service in mechanical engineering works. Although by far the greater part of his life was devoted to civil engineering, he fortunately approached that profession by five years of apprenticeship in the shops of the Caledon Engine Works in Dundee. Following this, and like many Scots before him, he hied to London, and served three years in the drawing offices of notable marine engineers and shipbuilders, the Thames Iron Works of Millwall, and Humphreys and Tennant of Deptford, both on the River Thames.

In 1895 he was appointed engineer-in-charge of construction on the Surrey Commercial Docks for the contractor, S. Pearson and Son, Ltd., with whom he remained twenty years, until 1915, which included his experience in the United States with that firm. His tunneling experience began as agent for S. Pearson and Son, Ltd., on one of the London tube railways, the Great Northern and City Railway. He came to the United States in 1904 when S. Pearson and Son, Ltd., appointed him a director and managing engineer of their New York, N. Y., office on construction of the group of four tunnels under the East River for the Pennsylvania Railroad Company as part of the latter's extension from Newark, N. J., to the City of New York.

In 1915 he was placed in charge of the production department of munitions in the United States by J. P. Morgan and Company, agents for the British Government, and, later, after the arrival of Mr. Balfour and the entry of America into the War, he continued in that position with the then constituted British War Mission to the United States. In this capacity his responsibility required rigid inspection at many factories, acceleration of output, assistance to contractors in overcoming difficulties, determination of work most suitable for particular companies, forecasting deliveries, and checking of shipments at the docks. For his service during this period of the war he was made a Commander of the Order of the British Empire in 1917, and in 1918 was raised to Knighthood of that Order on the deck of H.M.S. "Renown," anchored in New York Harbor, by the Prince of Wales, later the Duke of Windsor.

After the Great War he returned to England and became associated with engineering enterprises of magnitude, including the Prince of Wales Dock, Workington; King George V Graving Dock, Southampton; Dover Train Ferry Dock; and the jetty and power-house foundations for the Henry Ford Motor Works, Dagenham on the Thames.

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[1] Memoir prepared by James Forgie, M. Am. Soc. C. E.

In 1928, as chief engineer and works director, he joined the firm of John Mowlem and Company, Ltd., public works contractors of London, who constructed the aforementioned dock at Southampton, the largest in existence, and the train ferry dock at Dover. At the time of his death he was engaged in this capacity on the construction of two large reservoirs, each about a square mile in area, for the Metropolitan Water Board—one at Chingford and the other at Staines, on the northeast and southwest fringe of London, respectively. During his association with Mowlem and Company he contributed a paper on "New Methods of Excavating Quicksands and Stable Ground" to the Societé des Ingenieurs Civils de France, and a paper on "Modern Methods of Plant for Excavation" to The Institution of Civil Engineers. In earlier years (1909) in New York he contributed a paper to the American Society of Civil Engineers on "Caisson Disease and Its Prevention."[2] In London in 1935 he became a member of the Joint Committee on Regulations for the Guidance of Engineers and Contractors for Work Carried Out Under Compressed Air, set up by The Institution of Civil Engineers and The Federation of Civil Engineering Contractors. The committee published its report in January, 1936; a report somewhat different in its proposed limitations to those of American State labor codes. Sir Henry devoted much time and study to this subject, of which he had had varied and successful practical experience in both countries.

Shortly before his decease he collaborated with two other engineers in preparing a report to the Lord Privy Seal on air-raid shelters, published as a "White Paper" in February, 1939.

He was a member of The Institution of Civil Engineers and of the Engineering Institute of Canada. The Whitworth Society (England) elected him its president for the year 1936–1937. In London he maintained Scottish associations through membership in the Caledonian Society, the Burns Club, the Royal Mid-Surrey Golf Club, and the Caledonian Club, the membership of which is confined to men of Scottish descent. (The writer, also a member of the Club, has recently received advice from its house committee stating that "as a result of enemy action the Club premises have been damaged to such an extent as to be unfit for occupation" and to accept "in the meantime" the hospitality of two clubs in the vicinity.) Sir Henry served on the committee of this Club for a number of years. At the Royal Mid-Surrey Golf Club he pursued the "wee bit ba" with much zest; the writer when playing with him in September–October, 1938, used a cleek (iron club) given him on that same course in 1895. During this period of the Munich crises Sir Henry's firm, with a few thousand men, started digging bomb-shelter trenches in London open areas including this golf course near the Club house—the only noticeable evidences, physical or mental, of the threat of war in London and elsewhere in England and Scotland.

Sir Henry was married twice. His first wife, the former Elizabeth Hodge of Dundee, by whom he had two sons and two daughters, died in 1911. Surviving him are one son and two daughters and his second wife, the former Kathie Sutherland of Montreal, to whom he was married in 1915.

Sir Henry Japp was elected a Member of the American Society of Civil Engineers on May 3, 1905.

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[2] *Transactions*, Am. Soc. C. E., Vol. LXV (1909), p. 1.

LEONOR FRESNEL LOREE, M. Am. Soc. C. E.[1]

DIED SEPTEMBER 6, 1940

Leonor Fresnel Loree was born in Fulton City, Ill., on April 23, 1858, the son of William Mulford and Sarah Bigelow (Marsh) Loree. He was graduated from Rutgers University, in New Brunswick, N. J., in 1877, with the degree of Bachelor of Science. He received the degrees of Master of Science, Civil Engineer, and Doctor of Laws in 1880, 1896, and 1917, respectively.

Mr. Loree entered the service of the Pennsylvania Railroad Company in 1877 as assistant in the engineer corps. This was the beginning of a long career in railroad work, which ultimately led to his becoming president of The Delaware and Hudson Railroad Company in 1907. During the intervening years, he held many positions, which laid the foundation for the latter work. These included:

1877 to 1881—Transitman, engineer corps of the U. S. Army.
1881 to 1883—Leveler, transitman, and topographer on preliminary survey and location of Mexican National Railway from Rio Grande River to Saltillo, Mexico.
1883 to 1884—Assistant engineer, Chicago (Ill.) Division of Pennsylvania Lines west of Pittsburgh, Pa.
1884 to 1886—Engineer of maintenance of way, Indianapolis and Vergennes Division.
1886 to 1888—Engineer of maintenance of way, Chicago Division.
1888 to 1889—Engineer of maintenance of way, Cleveland (Ohio) and Pittsburgh Division.
1889 to January 15, 1896—Superintendent of Cleveland and Pittsburgh Division.
January 15, 1896, to January 1, 1901—General manager of Pennsylvania Lines west of Pittsburgh.
January 1, 1901, to June 1, 1901—Fourth vice-president of Pennsylvania Lines west of Pittsburgh.
June 1, 1901, to January 1, 1904—President of Baltimore and Ohio Railroad Company.
January 1, 1904, to October 4, 1904—President of Rock Island Company of New Jersey, and chairman of Executive Committee of Chicago, Rock Island and Pacific Railway Company and St. Louis–San Francisco Railway Company.
June 15, 1906, to December 31, 1936—Chairman of Executive Committee, Kansas City Southern Railway Company.

After leaving the Chicago, Rock Island and Pacific Railway Company, Mr. Loree did consulting work for E. H. Harriman on various railroad matters until April 10, 1907, when he became president of The Delaware and Hudson Railroad Company, then known as the President, Managers and Company of The Delaware and Hudson Canal Company, and later as The Delaware and Hudson Railroad Company. Still later it became The Delaware and Hudson Company (the parent company) and The Delaware and Hudson Railroad Corporation.

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[1] Memoir prepared by James MacMartin M. Am. Soc. C. E.

It was when Mr. Loree came to The Delaware and Hudson Railroad Company in 1907 that the writer first met him and accompanied him on his trip over the property—first, from Albany, N. Y., to Wilkes-Barre, Pa.; and then, a week later, from Albany to Montreal, Quebec, Canada. Afterward the writer went with him on all of his trips over the property until his retirement in 1938. From 1907 until 1917 the writer went over engineering matters with him each Monday at his office in New York, N. Y. It was during that period that the writer had the opportunity of observing him as a railroad officer and, the writer thinks, as a friend. He was sparing of speech, a good listener, and his eyes were always open; and when he did speak, he had something worth while to say. His knowledge of railroading and everything pertaining thereto was inexhaustible, as was his knowledge of other matters, and it was a pleasure to listen to him converse.

Mr. Loree has been referred to of late years as one of the last of the old-time railroad officials. That, he was! He belonged to the class of railroad men who learned their railroading the hard way, as is illustrated by this synopsis of his career. He was one of the class of railroad men who, by their hard work, practice of economy, and use of "horse sense," have made the railroading of today possible.

When Mr. Loree was made president of The Delaware and Hudson Company in 1907, its financial condition was critical. Approximately $26,000,000 had been borrowed, mostly on short-term notes, and his first problem was to fund this debt. He completed the funding during 1907 and was able to continue the payment to stockholders of the 9% dividends (which rate had been put into effect on January 1, 1907) until July 1, 1932, when it was reduced to 1% for the remainder of the year. The company ceased paying dividends on January 1, 1933, a long time after most other roads had been compelled either to reduce dividends or had ceased paying them entirely. The stockholders of the company were fortunate to have had as president a man who could accomplish this.

In 1905, about two years before Mr. Loree came with it, The Delaware and Hudson Company had purchased the Quebec, Montreal and Southern Railway, extending from St. Lambert, Quebec, Canada (on the east side of the St. Lawrence River), to Pierreville, Quebec, Canada, with trackage rights over the Victoria Bridge into the Grand Trunk Station at Montreal, a distance of 62 miles, which was known as the Shore Division. There was also what was known as the Southern Division, extending from Noyan Junction to Sorel, via Bellevue Junction (all in Quebec, Canada), and then using the Shore Line to Pierreville, a distance of 82 miles, a total main line trackage of 144 miles, with sidings of 13.03, or a total mileage at that time of 157.03.

Contracts had been let, after purchase, to extend this road from Pierreville easterly 48.2 miles to Ste. Philomène, Quebec, Canada, located on the Lotbiniere and Megantic Railroad about 7 miles west of St. Jean des Chaillons, Quebec, Canada, and this construction was about 70% completed when Mr. Loree came to the company. After going over the project, Mr. Loree was in favor of suspending this work, but it was finally decided that the cost of doing so would be more than that of completing it, and therefore it was completed and operated for some years. Afterward, Mr. Loree succeeded in selling it to the Canadian National Railway Company at a price which more than recompensed the company for what they had expended on it.

When Mr. Loree came to the Delaware and Hudson Railroad Company in 1907, the facilities for engine and car repairs and construction were in extremely poor condition, being antiquated and scattered over the property. He studied this situation and the result was the building of the present shops at Colonie, N. Y., at a cost of $2,600,000. This resulted in a large saving of money and the quickening in time of engine and car repairs and construction.

Prior to 1914, the connection between The Delaware and Hudson Company and the Pennsylvania Railroad Company at Wilkes-Barre was made over the Lehigh Valley Railroad, using their tracks through the City of Wilkes-Barre over many grade crossings, with the resultant loss of considerable time and money. After thorough investigation, it was decided to build a connecting line along the west side of the city, and so the Wilkes-Barre Connecting Railroad was built at a cost of approximately $2,600,000 by The Delaware and Hudson Company and the Pennsylvania Railroad Company, each owning 50%. The building of this railroad resulted in a great saving in money and time, and the writer believes that The Delaware and Hudson interest in this company is one of the most valuable assets it has.

These three things—the selling of the Quebec, Montreal and Southern Railway, the building of the shops at Colonie, and the building of the Wilkes-Barre Connecting Railroad—are, in the opinion of the writer, the most valuable engineering done by Mr. Loree on the railroad. Of course he completed many other projects during his term of office, but most of them are well known and are not pertinent here. They can be found in the *Biographical Directory of the Railway Officials of America* for 1906 and 1922; in *Who's Who in The Railroad in North America* for 1930 and 1940; and in articles appearing in the *Railway Age,* April 9, 1938, and September 14, 1940, which describe Mr. Loree's career generally.

In addition to his technical ability there was another side to the character of Mr. Loree that is not known to any but his friends and those who have benefited by it. He was a kindly man, and there is many an employee of The Delaware and Hudson Company, in every class of the service, who can testify to that fact.

A classmate of Mr. Loree, Howard N. Fuller, when in Rutgers University, wrote a bit of verse which has been quoted many times in print, and which applies to Mr. Loree as the writer knew him better than he can express it. It follows:

"So may I live that when I die
My life may show no blot of shame.
And o'er the grave wherein I lie,
Upon a low and modest stone,
Where all that pass may quickly scan
May this be writ, and this alone:
'He never wronged his fellowman.'"

On January 29, 1885, Mr. Loree was married to Jessie Taber who, with three children, James Taber, Robert Fresnel, and Louise (Mrs. David M. Collins), survives him.

Mr. Loree was elected a Member of the American Society of Civil Engineers on October 6, 1886.

GEORGE JOHN LYON, M. Am. Soc. C. E.[1]

Died December 17, 1939

George John Lyon was born on June 8, 1874, in New York, N. Y., the son of George Lamson Lyon and Fannie Mary (Crowe) Lyon. He was an eighth-generation descendant of Thomas Lyon of Rye, N. Y., who arrived in the Massachusetts Bay Colony about 1640.

He was educated in the public schools of New York City. In 1899 he was graduated from the general science course of the University of Nebraska, at Lincoln, Nebr., with the degree of Bachelor of Science. In 1904 he received the degree of Civil Engineer from the School of Mines, Columbia University, New York City, and in 1916 he obtained the Civil Engineer degree from the University of Nebraska. He held no honorary degrees.

Following his graduation from the University of Nebraska in 1899, Mr. Lyon was employed as rodman, topographer, and instrumentman for the location and construction of the yards and depot of the Union Pacific Railroad Company at Omaha, Nebr. The next year he was inspector of dock construction and instrumentman for the New York Central Railroad Company during the construction of an ore dock at Buffalo, N. Y.

During 1901 and 1902 Mr. Lyon was junior assistant engineer of the New York, Ontario and Western Railway Company for the installation of second track and maintenance of way. Between October, 1902, and June, 1904, he was a student at Columbia University, graduating with the degree of Civil Engineer.

From June to September, 1904, he was a structural draftsman with Post and McCord, Structural Engineers of New York City, detailing structural steelwork for the St. Regis Hotel, New York.

This varied experience was invaluable to him in his new position as professor of civil engineering at Colorado College, Colorado Springs, Colo., which he held from September, 1904, to July, 1910. While at Colorado College Professor Lyon organized and headed the Department of Civil Engineering, organized and directed the Summer School of Engineering, and installed the hydraulic and material testing laboratories. In addition to his curricular duties he was regularly engaged in professional work during vacation periods for the purpose of extending his engineering experience.

In the summer of 1905 Professor Lyon worked as bridge draftsman in the Trenton, N. J., plant of the American Bridge Company, detailing steelwork for the Pennsylvania Railroad's New York City passenger terminal. In 1906 and 1907 his free time was given to an investigation of the strength of cable sleeve joints for copper transmission cable, and other details of transmission lines, and of the strength of concrete, for the Central Colorado Power Company. During the summer of 1907 he served as engineer for the Citizens' Water Committee of Colorado Springs, investigating the utilization and conservation of the domestic water supply.

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[1] Memoir prepared by Rolston Lyon, Esq., Washington, D. C.

On July 1, 1908, Professor Lyon was appointed field assistant, water Resources Branch, U. S. Geological Survey. He made an investigation of, and a report on, the majority of the hydroelectric power plants in the State of Colorado for the Federal Power Census. For the summer and fall of 1909 he was chief engineer of the Platte River Reservoir Company, in charge of surveys, maps, and filings for the $250,000 project which included a dam, a reservoir, and irrigation ditches.

Professor Lyon was appointed assistant engineer, Water Resources Branch, U. S. Geological Survey, on May 2, 1910. This appointment was continued in force until August 2, 1917, when it was "discontinued without prejudice" because of his commission in the Corps of Engineers, U. S. Army. During the summer and fall of 1910 he was special deputy state engineer, State of Colorado, on a commission to investigate seepage and evaporation losses in the South Platte River, and he investigated and reported on the 250,000-acre Turley Irrigation Project, San Juan River, Colorado, adjoining the Navajo Indian Reservation in northwestern New Mexico, for the U. S. Geological Survey.

From September, 1910, through July, 1916, he was assistant professor of Civil Engineering at Union College, Schenectady, N. Y., where he organized and directed the summer field work in surveying. As at Colorado College, Professor Lyon continued his consulting work during his free periods. In the summer of 1911 he made a water-power census of the State of Connecticut for the U. S. Geological Survey, and supervised the preparation of stream-bed profiles in the Connecticut River watershed.

During the summer of 1912 he designed and constructed the Spiers Falls (N. Y.) Automatic Recording Gaging Station on the Hudson River. This is the base station, to the records of which are related all of the hydrological data for Upper Hudson drainage. In the following two years he made hydrologic studies of the Hudson, Mohawk, Black, Saranac, Sacandaga rivers, and smaller streams, and erected several standard stream gaging stations in New York State. He prepared flood studies, and temperature and precipitation studies for the State of New York, made an investigation of the design and construction of certain locks on the New York State Barge Canal, and gave general attention to all types of canal structures.

"Plans and Specifications for Current-Meter Gaging Stations," by C. C. Covert, Assoc. M. Am. Soc. C. E., G. J. Lyon, and C. H. Pierce, M. Am. Soc. C. E., was published by the U. S. Geological Survey, Washington, D. C., in 1913.

During the summer of 1914 Professor Lyon qualified as an expert hydraulic engineer and testified before the Public Service Commission of New York concerning hydroelectric developments of the Mongaup Heat, Light and Power Company. Later that year he qualified as an expert engineer before the New York State Court of Claims in the $500,000 suit "Saratoga Springs Reservation vs. Lincoln Springs Company," and in the $2,000,000 suit "Rumsey Company, Seneca Falls, vs. the State of New York."

Professor Lyon's book, "Equipment for Current-Meter Gaging Stations," was published in 1915 as *Water Supply Paper No. 371* by the U. S. Geological Survey, Washington, D. C.

From 1916 to 1917 Mr. Lyon was engaged as consulting hydraulic engineer by the Marseilles Land and Water Power Company, Ottawa, Ill., for which he made an intensive study of the hydraulics of the Illinois River in connection with a $2,000,000 suit over hydroelectric power and water rights at the Marseilles dam. Later, in 1920 and 1921, he was called as a witness in that suit, and was accepted by the Circuit Court of Illinois as an expert hydraulic engineer. His continuous testimony for forty-eight consecutive court days established a record for the State of Illinois.

The Imperial Russian Government consulted Mr. Lyon in the summer of 1917 with respect to stream gagings in Turkestan. He also prepared, for the Terry Shipbuilding Corporation of New York and Savannah, estimates covering the cost of municipal improvements necessary for the construction of their industrial village at Port Wentworth, Ga., in connection with the Wooden Ship war project.

A "Manual of Hydraulic Engineering Instruments," by Geo. J. Lyon, on which he had worked during 1917, was published in 1918 by the W. and L. E. Gurley Company of Troy, N. Y. In addition to the three books mentioned previously, Mr. Lyon was, at various times, a contributor to the *General Electric Review*, the *Stone & Webster Journal*, the *School of Mines Quarterly* of Columbia University, and the *Nebraska Blueprint* of the University of Nebraska.

Mr. Lyon's education at the University of Nebraska, a land-grant college, had included four years of drill and four years of class work in Military Science and Tactics under the instruction of Lieut. (later General) John J. Pershing. In his senior year at the university Mr. Lyon commanded an artillery platoon. When the United States entered the World War he sought to find how he could best serve his country. He was offered commissions in both the Ordnance and Quartermaster Corps of the Army, but preferred to serve with that branch of the Army for which his professional training best suited him, although such a choice was made at the cost of a lower rank.

On June 19, 1917, he was commissioned Captain in the Engineer Officers Reserve Corps. On August 3, 1917, he was assigned to active duty in the Construction Division of the Army as Captain of the Corps of Engineers. His first duty, which lasted until the end of the year, was as engineer assistant to the constructing quartermaster at Camp Hancock, Augusta, Ga. There he had charge of the construction of 25 miles of water supply piping varying in size from 16 in. to 6 in., and of an equal mileage of service pipes. He superintended the construction of sanitary facilities, sewers, etc., for the camp, the Base Hospital, and the Remount Station. It is a matter of official record that "No unit of the 28th Division, which was the first division to be completely mobilized for service, ever entered the area assigned to it without finding the water supply in working order on its arrival."

Between January and May, 1918, Captain Lyon was engineer assistant to the constructing quartermaster at the Nansemond Ordnance Depot (Pig Point), Norfolk, Va. His supervision of the construction of a water works at that station included the sinking of deep artesian wells and the installation of the necessary pumping machinery and accessories.

He returned to Schenectady, in May, 1918, as engineer assistant to the constructing quartermaster at the Schenectady Quartermaster Interior Storage Depot. There he built and equipped two warehouses 160 by 1,400 ft in size, six warehouses 160 by 1,680 ft, two open storage sheds of similar dimensions, a railroad yard containing eleven miles of track, and the necessary housing facilities for guard and labor battalions, including water supply and other utility systems. While assigned to Schenectady Captain Lyon was also engaged in special work for the Intelligence Division of the Army.

From March to May, 1919, he had charge of the construction of quarantine stations at Savannah, Ga., Hampton Roads, Va., and Delaware Bay, Del. Between May and July of that year he was engineer assistant to the constructing quartermaster, Pedricktown Ordnance Depot, Pedricktown, N. J., where he superintended the completion of powder and ammunition magazines, wharves, piers, and railroad facilities on the Delaware River similar in magnitude to the work at Schenectady.

From July, 1919, to March, 1920, Captain Lyon was assigned to various locations in the Hampton Roads area of Virginia for duty in connection with the water supplies of Fort Monroe, the Army Base at Norfolk, the Navy Yard at Portsmouth, Suffolk, and Big Bethel. Part of his duties included the adjustment of contracts between private water companies and the government. In his official capacity he assisted the private companies in the enlargement and standardization of their water supplies as required for army purposes. Acting for the government, he was in charge of land surveys and the purchase of reservoir sites.

An investigation of water rates and tests of the fire protection facilities caused Captain Lyon's assignment to the New Cumberland Interior Storage Depot at Harrisburg, Pa., between March and May of 1920. During that time he also appeared before the Public Service Commission of Pennsylvania as an expert hydraulic engineer for the United States in its suit against the Riverton Consolidated Water Company. In June he returned to Hampton Roads, where he was in charge of the sale of parts of the water works developments by the Army to private corporations.

Captain Lyon requested an immediate discharge from the Army on August 12, 1920, to re-enter private consulting practice, and on August 17, 1920, he was honorably discharged from the Corps of Engineers. He proceeded at once to Ottawa, Ill., where, as mentioned previously, he continued his work for the Marseilles Land and Water Power Company.

Following this he was engaged for the next three years (1922 to 1925) by the Interstate Commerce Commission, Washington, D. C., in the engineering examination of applications by railroads for loans, and for permission to issue new securities. In 1925 he was retained for the appraisal of four gas plants in the vicinity of Harrisburg, in connection with their consolidation.

In 1926 Mr. Lyon was engaged in an efficiency study of the entire organization of the Dorr Company, Inc., manufacturers of specialized mining and separating machinery. He made similar studies of the operations of Kneeland's Motor Express, Inc., a New England trucking company, and of the Elk River Clay Products Company in West Virginia.

From 1927 until his death Mr. Lyon was engaged in the capacity of hydraulic engineer by the National Board of Fire Underwriters in New York City. During this period he examined the water supplies of more than one hundred cities in the United States, from Maine to Texas, which had populations of 50,000 or more, including New York City. These examinations were to determine the capacity and availability of city water supply systems with reference to fire protection, and included exhaustive investigations of the source, the purification, and the distribution of the water. Recommendations made in published reports were then used as a basis for the determination of fire-insurance rates by the various rating agencies.

Mr. Lyon was a member of Gamma Chapter of Delta Phi fraternity, and in 1915 he was elected to honorary membership in Alpha Chapter, at Union College, in recognition of his part in the preparation of plans and specifications for, and the construction of, the new Chapter House while he was a professor at the College.

In May, 1904, he was elected to membership in Kappa Chapter of the Society of the Sigma Xi, while a student at the Columbia University School of Mines.

His membership in academic organizations included the University of Nebraska Alumni Association, the University of Nebraska Alumni Club of Washington, D. C. (of which he was President in 1924), and the Columbia Alumni Association. In his professional field Mr. Lyon held memberships in the Winter Night Club (Colorado Springs, 1904–1912); the Colorado Polytechnic Society (Colorado Springs, 1904–1910), of which he was secretary from 1906 to 1910; the Society of Engineers of Eastern New York, 1912–1920; the American Association for the Advancement of Science; and the Washington Society of Engineers. On November 21, 1921, he was elected to membership in the Cosmos Club of Washington, D. C., in which he retained membership until his death. He was also a member of the Columbia University Club in New York City. Mr. Lyon was elected a member of the American Institute of Mining and Metallurgical Engineers in 1914.

A man of acknowledged ability in his field, and of the highest integrity in every phase of his life, Geo. J. Lyon exerted his widest influence through his teaching. He was actively engaged throughout his career in acquiring information in all branches of his chosen profession which he was ready to share with others who asked his advice. He gave to his work thoroughness, honest endeavor, his most careful consideration, and his best judgment.

For recreation Mr. Lyon studied the military tactics of the Civil War, and railroad history. Whenever his work took him into the battle areas his training enabled him to clarify his own opinion on controversial points of war history. He was extremely well informed on the corporate and operating histories of most of the North American railroads, following their changes in policy with great interest.

His most important personal research, unfortunately not completed at the time of his death, was an engineer's study of his American ancestry. From old deeds, town surveys, and other official records he was mapping the land holdings and migrations of each of the seven preceding generations of his family. This style of genealogical record was originated by Mr. Lyon because he believed that the landmarks and boundaries specified in the grants and deeds of early America were being obliterated by present-day highway and housing operations.

Mr. Lyon was married to Mary Rolston Coles of Mamaroneck, N. Y., on August 1, 1906, in St. Thomas' Episcopal Church. His widow and a son, Rolston, survive him. He was a lifelong member of the Protestant Episcopal Church.

Mr. Lyon was elected a Member of the American Society of Civil Engineers on June 24, 1914.

HARRY SHERWOOD ROYDEN McCURDY, M. Am. Soc. C. E.[1]

DIED AUGUST 13, 1940

Harry Sherwood Royden McCurdy, the son of James G. McCurdy and Mary S. (McLaughlin) McCurdy, was born in Moncton, New Brunswick, Canada, on September 15, 1876. His parents moved to Boston, Mass., when he was six years old, and he attended the public schools of that city. In 1894 he was graduated from the English High School where he was a captain in the High School Battalion.

After graduation, Mr. McCurdy entered the employ of the Metropolitan Park Commission and, as rodman, transitman, and assistant engineer, was engaged on topographical and real-estate surveys, and boulevard and parkway construction. At the same time he was taking a correspondence course in civil engineering.

In September, 1900, Mr. McCurdy was appointed a transitman with the Boston Transit Commission and for two years was in charge of lines and grades on the East Boston tunnel, being built for electric cars under Boston Harbor. In 1903 he was appointed assistant engineer and was placed in charge of a part of the construction of that tunnel, including stations and underpinning of buildings. This was followed by a period in the office of the Commission on estimates and designs for the location and construction of the Washington Street tunnel.

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[1] Memoir prepared by Carleton E. Davis, M. Am. Soc. C. E.

In March, 1906, Mr. McCurdy joined the staff of the Board of Water Supply of New York, N. Y., as assistant engineer in the Reservoir Department, where surveys, borings, investigation of sand deposits, borrow pits, character of earth available for embankments, quarries, and similar preliminary works were then in progress prior to the preparation of plans and letting of contracts for the construction of the main dams of the Ashokan Reservoir.

When construction on these features began in 1908, he was appointed division engineer in charge of the work, including the Olive Bridge Dam, the Beaver Kill dikes, the dividing and waste weirs, the upper gate-chamber and adjacent channels, and the pressure aqueducts from the upper gate-chamber to the lower gate-chamber. At a later date, when work was started on the headworks of the Catskill aqueduct and the aerator, Mr. McCurdy's responsibility was expanded to include the supervision of this work.

The basin of the Ashokan Reservoir, on Esopus Creek, containing about 127 billion gallons, and covering about 8,000 acres, is formed by the main dams and three smaller dikes closing gaps at the easterly end of the basin. The main dams, built under Mr. McCurdy's immediate supervision, consisted of the Olive Bridge Dam, 250 ft high above the bottom of the cutoff trench in the bed of the creek, a masonry structure spanning the gorge of the stream and flanked by earth wings; the Beaver Kill dikes, a continuous earth embankment 4 miles long, retaining a maximum depth of water of about 100 ft; a dividing weir, separating the reservoir into two basins; a waste weir and spillway channel; and channels, gate-chambers, and pressure aqueducts for the release of water from the reservoir to the aerator and headworks of the Catskill aqueduct. These structures required approximately 2,600,000 cu yd of excavation, 6,500,000 cu yd of embankment, and 1,000,000 cu yd of masonry.

The engineering organization of the Board of Water Supply, under the inspiring leadership of the late J. Waldo Smith,[2] Hon. M. Am. Soc. C. E., was notable for enthusiasm, unity, and devotion to ideals. Much of this was developed by local and interdepartment activities, in which Mr. McCurdy entered with the same energy that he gave to his direct engineering work.

In October, 1914, when work upon the Ashokan Reservoir was nearing completion, Mr. McCurdy resigned and entered the employ of the Miami Conservancy District, when the flood-control work for the Miami Valley, in Ohio, was in the investigation stage and when plans for design and construction of the project were being organized. For several years during this investigation stage he was assigned to the examination of the dam sites and investigation and analysis of borrow-pit materials for embankments and concrete aggregates.

In January, 1918, he was appointed division engineer in charge of construction of the Englewood Dam, an hydraulic fill structure about 4,700 ft long, with a maximum height of 125 ft, containing 3,600,000 yd of embankment and 25,000 yd of concrete in spillway and outlet structures. The Englewood Dam was the largest and highest of the five flood-control dams built by the District. It was constructed by District forces, was completed at less than estimated cost, and was ready to control floods about one year ahead of schedule.

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[2] For Memoir, see *Transactions*, Am. Soc. C. E., Vol. 101 (1936), p. 1502.

Upon the completion of construction at the Miami Conservancy District in the spring of 1923, Mr. McCurdy was engaged by the Highway Commission of Manchester, N. H., and for a year following was in charge of the work of that department with the title of surveyor. During this time the Queen Street Bridge, spanning the Merrimack River, and a main outfall sewer were constructed under Mr. McCurdy's general supervision.

After the completion of his term of service with the City of Manchester, Mr. McCurdy, during 1925 and a part of 1926, was construction engineer for the firm of Patrick McGovern, Inc., contractors for the northerly part of the subway under Broad Street, in Philadelphia, Pa.

In August, 1926, he became chief engineer of the Philadelphia Suburban Water Company and continued as such until his death. During this period the company, under his supervision, constructed its Pickering Creek Reservoir, with an Ambursen-type dam, about 35 ft high; and its Springton Reservoir, with an earth embankment about 70 ft high, together with additional filtration plants, enlarged pumping stations, and several hundred miles of pipe lines. At the time of his death, Mr. McCurdy had made substantial progress in the preliminary investigations, surveys, borings, and designs for a proposed reservoir on Ironworks Creek, with a dam to consist of an earth embankment, holding about 35 ft of water.

As chief engineer of the Philadelphia Suburban Water Company, Mr. McCurdy was intimately associated with the development of the company's plant inventory; its valuation; its maps, plans, and records; the operation of its purification plants; and the numerous engineering features of routine operation. As an incident, he designed and supervised the construction of the filtration and softening plant of the City of Boca Raton, Fla., and the private water supply of the Boca Raton Club. He was consulted by the Indianapolis (Ind.) Water Company in connection with dams and purification plants, planned or proposed by that company.

Mr. McCurdy was a member of the New England Water Works Association and the American Water Works Association. He was fond of golf, finding in that pursuit his chief outdoor relaxation, and was an honorary member of the Seaview Golf Club. He was fond of meeting people and found contact with others stimulating and inspiring. This characteristic led him to develop a series of illustrated talks on the "Origin and Development of Public Water Supplies," which were in demand in the schools of suburban Philadelphia, and which were practically a part of the regular curriculum in many of them.

In 1912, Mr. McCurdy was married to Mary C. Jordan, of Shippensburg, Pa., who, with three children, Richard C., Elizabeth, and Janet, survives him.

Mr. McCurdy was elected an Associate Member of the American Society of Civil Engineers on September 7, 1904, and a Member on April 30, 1912.

HOOD McKAY, M. Am. Soc. C. E.[1]

DIED FEBRUARY 1, 1940

Hood McKay was born in New York, N. Y., on February 29, 1868, the son of Henry McKay and Henrietta (Allen) McKay. He was educated at Germantown Academy, Philadelphia, Pa., and St. Paul's School, Concord, N. H.

He was employed from 1886 to 1912 by the Philadelphia and Reading Coal and Iron Company, the Mineral Railroad and Mining Company, Lykens Valley Coal Company and Affiliates, and the Lehigh Coal and Navigation Company. He served as engineer and superintendent in development work, active mining operations, and in executive capacities.

From 1912 through 1914 he was engaged in private practice as a consultant on coal mining matters in the western part of the United States and British Columbia. He also served as consultant during this period for various clients interested in gold, silver, and copper mining in the western part of the United States.

Hood McKay, as president of the O. S. Richardson Coal Company from 1914 to 1918, developed facilities for wholesaling anthracite fuel in Chicago, Ill. The company, under his leadership, developed dock facilities for water-rail distribution of coal in this area, and the necessary equipment for handling the fuel expeditiously and economically.

From 1919 until his retirement on March 1, 1939, he served as vice-president and general manager of the Susquehanna Coal Company and allied interests.

Hood McKay's connections with the anthracite coal industry spanned the major part of the life of that industry in the United States. He had a broad experience and a profound knowledge of the responsibilities and possibilities of the location, development, operation, and distribution of the fuel, and was considered by his associates as an expert on many phases of his life work. Like most men growing up with an industry and functioning as a pioneer, he retained to the end an aggressive but practical attitude toward coal mining, which was both his vocation and avocation.

He was married to Bertha Weineck of Lykens, Pa., on September 20, 1915, who survives him. At the time of his death, they were residing in Philadelphia, Pa.

Hood McKay was elected a Member of the American Society of Civil Engineers on November 6, 1907.

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[1] Memoir prepared by Joseph A. Russell, Assoc. M. Am. Soc. C. E.

THOMAS KNIGHT MATHEWSON, M. Am. Soc. C. E.[1]

DIED MAY 2, 1940

Thomas Knight Mathewson, the son of John Arnold Mathewson and Mary Brown (Knight) Mathewson, was born in Muscatine, Iowa, on March 10, 1870. The ancestors of both parents arrived in America during the colonial period early in the seventeenth century. On the paternal side they came from England, and those on the maternal side were Huguenots from France. Rhode Island was the native state of Mr. Mathewson's parents who were married on November 10, 1857, and went west immediately afterward to Muscatine, which became their permanent home. His father was a civil engineer as is also his elder brother, Isaac Mathewson.

Mr. Mathewson received his early education in the public schools of Muscatine. He was graduated from high school in 1888. The following year was spent at additional studies before entering the University of Michigan, in Ann Arbor, Mich. He received the degree of Bachelor of Science in Civil Engineering from that university in 1895. While a student he gained considerable experience during vacations working with his father and brother on various projects.

After graduation he was employed for six months by the United States Government as assistant inspector on the Mississippi River Improvement, laying out and constructing wing dams and shore protection, brush mattresses, and riprap north of St. Louis, Mo. His next engagements were as levelman on railroad location out of Rock Island, Ill.; assistant city engineer of Muscatine; and assistant engineer of maintenance, Burlington, Cedar Rapids and Northern Division of the Rock Island Lines.

Following these engagements most of his professional career was spent in foreign countries. He went to Juarez, Mexico, as engineer in charge of terminals of the Rio Grande, Sierra Madre and Pacific Railroad Company in 1897 and continued in that position for two years. The work consisted, in part, of layout and construction of yards, roundhouse, and pile bridge over the Rio Grande to El Paso, Tex.

In 1898 he went to Cuernavaca, south of Mexico City, to become assistant engineer for the Cuernavaca and Pacific Railroad on location, construction, and tracklaying and in charge of party on survey and exploration for extension down the Balsas River to the Pacific Coast. This engagement lasted two and one-half years.

From 1902 until 1914 Mr. Mathewson was connected with the Guanajuato Power and Electric Company in the development of hydroelectric power and its use in the central states of Mexico. The original development in this region was promoted for the purpose of aiding and reviving the mining industry at and near the City of Guanajuato, which had been known as a mining center for more than three hundred years. The first power site and concession was investigated and acquired under the guidance of the late H. H. Filley,

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[1] Memoir prepared by Irwin C. McBride, Esq.

M. Am. Soc. C. E., who was chief engineer and general manager during the initial stages of the formation of the Guanajuato Power and Electric Company. In 1902 Mr. Mathewson, as locating engineer, made the preliminary surveys and plans for governmental approval of the Platanal Plant near Zamora, Michoacán. He was assistant in charge of construction the following year. This power plant, 100 miles from Guanajuato, is connected with that city by a steel tower transmission line, for which Mr. Mathewson made the location survey. This steel tower line was the first ever built for high tension—60,000 volts. The Aermotor Company's windmill towers were adapted to this use.

In 1905 Mr. Mathewson transferred temporarily to assist in the layout and development of an ore-treating plant at Guanajuato for the Reduction Mines Company, which was to become one of the principal consumers of power.

In 1906, after an additional unit had been installed at Platanal with his assistance, Mr. Mathewson investigated two other power sites for the Guanajuato Company in the State of Michoacán near the town of Panindicuaro. These properties were acquired and plans for their development made under the title of The Michoacán Power Company. Following the death of Mr. Filley in 1907, Mr. Mathewson became chief engineer. Under his direction plans for both developments were made for government approval, and the construction was completed. These plants, Sabino and Botello, are on the Angulo River and develop a total head of 250 m. Although fed by springs at a fairly constant flow, additional water was needed for a part of the year. During 1910 and 1911 the drainage basin was surveyed and a favorable site selected for the storage of flood waters. Mr. Mathewson submitted a report that later became a basis for the construction of a dam to effect the storage desired.

During the period 1912 to 1914 he made studies, plans, and favorable reports upon other hydroelectric projects, none of which have been completed. By 1914 the revolution had disrupted all business completely, and he left central Mexico following the capture of Veracruz by forces of the United States in April of that year. At the time of the Veracruz incident he was in camp on the Lerma River near La Piedad, Michoacán, in the central part of the Republic. Americans were ordered out, but travel to the north was blocked. The only open route was by rail to Mexico City and then to Puerto México on the Isthmus of Tehuantepéc, from which place refugees were taken to New Orleans, La.

After completion of a report upon the work that he had left, his next engagement was with the Richardson Construction Company (of which the late Louis C. Hill, Past-President, Am. Soc. C. E., was construction engineer) in charge of field work on the Yaqui River Canal, State of Sonora, Mexico. He then took a position with the Southern Pacific Lines and was on the Southern Division at Empalme de Cañitas and the Sinaloa Division at Mazatlán replacing destroyed bridges and ballasting, adding flood protection, and doing general maintenance work until revolutionary troubles and Yaqui Indian raids interrupted the work in 1916.

In 1917 he went to Cuba as assistant engineer for the Northern Railroad Company where he was in charge at the front on the construction of a new line and, later, on reconstruction following wreckage by the Gomez revolution.

From Cuba he went to Santiago, Chile, in 1917, where he became chief engineer of the National Power and Electric Company, which was organized to construct hydroelectric developments near that city. He planned and mainly constructed the Maitenes plant of 20,000-hp base load with pumped storage for a peak load of 35,000 hp. This plant drives the trains on the State Railway and Transandine line to Argentina and serves the commercial load at Valparáiso, Chile, and other points. He also planned the Queltehues development for a base load of 50,000 hp, which was later constructed as projected by him.

After five years on this work in Chile, near Santiago, an almost fatal illness obliged him to retire from active engineering in 1923. After his retirement he continued to live in Santiago until July, 1930, when he returned to the United States and located in Santa Ana, Calif., where he resided until his death on May 2, 1940.

While working in Mexico, Mr. Mathewson was a member of the American Club of Mexico City. During his residence in Chile he was a member of the American Club and of the September Club of Santiago. He was a life member of both the University of Michigan Alumni and the National Geographic Society, and a member of the Roman Catholic Church.

He was married on May 18, 1922, in Santiago, Chile, to Mrs. Edith Mary (Walker) Brede, who, with their son, Thomas Walker Mathewson, survives him.

Thomas Knight Mathewson was an able engineer. Careful and thorough in planning and design, he gained the high regard and appreciation of employer and co-worker. Although of a quiet and retiring disposition, he made many close friends. His personal letters were clever and entertaining—comments of the bystander as he saw the world go by. He was a devoted husband and father. To have known him was a privilege.

Mr. Mathewson was elected a Member of the American Society of Civil Engineers on May 1, 1907.

CHARLES FILLMORE MEBUS, M. Am. Soc. C. E.[1]

DIED JUNE 28, 1940

Charles Fillmore Mebus was born on January 7, 1865, in Palmer Township, Northampton County (near Nazareth), Pa., the son of William Andrew and Sucillia (Georg) Mebus. His early education was obtained in the local schools.

He spent one and one-half years at Valparaiso University, in Valparaiso, Ind., and entered Lafayette College, in Easton, Pa., in September, 1887. He studied engineering at Lafayette until June, 1890, and was then employed by the Lehigh Valley Railroad Company until September, 1890, when he entered Vanderbilt University, in Nashville, Tenn. He continued with the study of civil engineering at Vanderbilt and taught mathematics at the same time. He completed the course in June, 1891. His credits at Vanderbilt were accepted at Lafayette and he is classed as a graduate of Lafayette College, Class of 1891, holding the degree of Bachelor of Science. He won the Junior Mathematical Prize at Lafayette in his junior year.

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[1] Memoir prepared by George B. Mebus, Assoc. M. Am. Soc. C. E.

Following graduation, Mr. Mebus was employed in Nashville by the late Walter Gill Kirkpatrick,[2] M. Am. Soc. C. E., as transitman on the Cumberland Park Race Course and on highway work, until December, 1891. At that time he entered the Nashville District office of the U. S. Engineers as a draftsman, continuing in that capacity until July, 1892. He was next employed as a draftsman by the Chicago Bridge and Iron Works in Chicago, Ill., from July, 1892, until October, 1892. During this time he was working on structural designs for buildings for the Chicago World's Fair.

In October, 1892, Mr. Mebus moved to Philadelphia, Pa., where he was employed by the Quaker City Elevated Railway Company on the design of a proposed elevated railway on Market Street west of the Schuylkill River. This engagement was terminated by the company's failure to obtain a franchise.

In June, 1893, he was employed by Chester E. Albright who conducted an engineering office in Philadelphia. In 1901 the partnership of Albright and Mebus was formed and lasted until 1931. During the period of his association with Mr. Albright, Mr. Mebus supervised the design of many large real-estate developments and also the design of many sewerage systems, most of which were in Pennsylvania. The major sewerage systems were as follows: Lower Merion Township; Borough of Narberth; Cheltenham Township; Borough of Lansdale; Borough of Bridgeport; Borough of Jenkintown; Radnor Township; Borough of Souderton; Borough of Quakertown; Borough of Chambersburg; City of Martinsburg, W. Va.; Byberry State Hospital; Danville State Hospital; Warren State Hospital; Norristown State Hospital; Delaware County Home; Montgomery County Home; and Eagleville Sanatorium. He also was engaged by the City of Clarksburg, W. Va., to prepare and execute a program for completely reconstructing the street paving for the city.

During 1917 and 1918 he was engaged as a consultant on several housing projects for the Emergency Fleet Corporation to provide residences for the shipyard workers in the vicinity of Philadelphia and Chester, Pa. Entire villages were planned, all improvements installed, and homes built.

In 1931 the partnership of Albright and Mebus was dissolved, and Mr. Mebus continued with his own office in Glenside, Pa., until June 10, 1940. During this period the following projects were handled by him: Sewage treatment plants for the Borough of Souderton, Borough of Chambersburg, Radnor Township, and Hatboro-Upper Moreland Township; and a 2,000,000-gal-per-day water filtration plant for the Danville State Hospital.

Mr. Mebus was always interested in civic affairs and was a member of the Board of Commissioners of Abington Township, Pa., from 1913 to 1935, serving as president from 1914 to 1931. He was former president of the Pennsylvania Association of Townships of the First Class, and a former engineer member of the Advisory Health Board of the Commonwealth of Pennsylvania.

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[2] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1639.

Mr. Mebus was a member of the Engineer's Club of Philadelphia for many years, acting as vice-president from 1907 until 1913, and as editor of the *Journal* which was then published as a quarterly. For twenty-two years he held membership in the Union League of Philadelphia. He was also a member of the Pennsylvania Society, Sons of the Revolution; the Pennsylvania German Society; Northampton County Historical and Genealogical Society; the Old York Road Historical Society; Pennsylvania German Folklore Society; Pennsylvania Parks Association; Pennsylvania Academy of Fine Arts; New Jersey Sewage Works Association; Pennsylvania Sewage Works Association; American Concrete Institute; American Planning and Civic Association; American Public Health Association; American Public Works Association; and Philadelphia Board of Trade. Mr. Mebus was a founder member of Company B, Engineer Battalion, National Guard of Pennsylvania, in which he served one complete enlistment, receiving an honorable discharge on January 8, 1912.

He was married to Nellie Ramsdell Taylor, of Canisteo, N. Y., on December 4, 1895. He is survived by his widow; two daughters, Helen E. Mebus (Mrs. H. M. Doebler), and Cecilia E. Mebus (Mrs. W. L. Boswell); and two sons, George B. Mebus, Assoc. M. Am. Soc. C. E., and Chester W. Mebus.

Mr. Mebus was elected a Member of the American Society of Civil Engineers on May 2, 1906. On December 23, 1937, he was made a Life Member of the Society.

JOHN THOMAS MOCKLER, M. Am. Soc. C. E.[1]

DIED MAY 7, 1939

John Thomas Mockler, the son of James and Brigid (Burns) Mockler, was born in Buffalo, N. Y., on September 3, 1887.

His engineering education was received at the Towne Scientific School of the University of Pennsylvania, Philadelphia, Pa., from which he was graduated in 1910 with the degree of Bachelor of Science in Civil Engineering.

During his college undergraduate years, Mr. Mockler was employed in various capacities on the New York State barge canal and by the Lackawanna (later Bethlehem) Steel Company. Immediately after graduation he was employed as a rodman for the Highway Department of the State of New York. He continued with this department until January, 1911, when he left to become a member of the Bion Arnold Company, of Chicago, Ill., on public utility appraisal, largely in Buffalo and western New York.

He remained only a short time in this work and returned to the New York State Engineer's office in April, 1911, where he did field work. In October, 1914, he became assistant engineer on the New York State Blue Line Survey.

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[1] Memoir prepared by John H. Feigel, Assoc. M. Am. Soc. C. E.

From May to October, 1915, Mr. Mockler was in charge of field work on the construction of cement concrete roads in the New York State Highway Department. He left the state service in October, 1915, to enter the Engineering Division of the Department of Public Works of the City of Buffalo. For two years he was employed by the Division of Engineering, first as a transitman and later as assistant engineer on survey and construction of city pavements.

In October, 1917, he was commissioned a First Lieutenant of Engineers and after attending the second engineering training camp at American University, Washington, D. C., and Belvoir, Va., he was sent overseas with the 514th Engineers, American Expeditionary Forces, where he made an enviable record in completing difficult, extensive construction projects. After discharge from the Army he returned to the Division of Engineering as assistant engineer and later as first assistant engineer in charge of construction and design of bridges, docks, and river improvement work.

Later Mr. Mockler was placed in charge of supervision of engineering design and construction in the Division of Engineering for the City of Buffalo, and from 1932 as first assistant engineer occupied the position of acting city engineer. During the period 1929 to 1936, he was in direct charge of the construction of Bird Island Pier Extension, the municipal piers at the foot of Michigan Avenue, Commercial Street and Dock Street wharves, the Buffalo Yacht Club, and the naval militia basin improvement. These projects designed and constructed by him were major improvements of the City of Buffalo's water front. In addition to the construction on the Buffalo lake front Mr. Mockler planned and completed an elaborate program of river and bank protection and straightening of the Buffalo River. The project included various types of protection bulkheads, and involved the expenditure of several hundred thousand dollars. The ultimate result of this work was the elimination of the annual flooding in the southeastern part of the city. Also during this period (1929–1936) Mr. Mockler designed and supervised the construction of bank protection and consolidation with willow planting as well as deepening and straightening the course of Cazenovia Creek between Cazenovia Park and Buffalo River.

Mr. Mockler's work was characterized by the conscientiousness of the highest type of municipal engineer, and his good nature and even temper were manifest in all his undertakings. He was a devout Catholic and was active in the Knights of Columbus and his church, the Holy Family Roman Catholic Church.

He had many interests, both engineering and social. He was recognized as a local tournament golf player and was proficient at figure skating and swimming.

Because of ill health Mr. Mockler retired from city service on April 1, 1938, and after a lingering illness he died on May 7, 1939.

He is survived by his widow, Mrs. Grace (Muckler) Mockler, to whom he was married on October 1, 1919. He is also survived by three daughters—Jane, Peggy, and Ann Mockler.

Mr. Mockler was elected an Associate Member of the American Society of Civil Engineers on October 14, 1919, and a Member on October 14, 1930.

RALPH MODJESKI, M. Am. Soc. C. E.[1]

DIED JUNE 26, 1940

Ralph Modjeski was born on January 27, 1861, in Cracow, Poland. His first American home was in California, and it was in that state that he died on June 26, 1940. He was the son of Gustav Sinnmayer Modrzejewski and Helena (Opid) Modrzejewski, who later became known as Modjeska, the première tragedienne of her time. It was as his mother's advance agent and secretary that Ralph came to America in 1878, thinking that he also would pursue his own artistic talents which he expressed through the medium of the piano, as the leading exponent of Chopin. He had been competently trained by Casimir Hofmann, son of the famous Josef Hofmann, and it was understood that Ralph Modjeski would follow the life of a professional pianist. However, his engineering instincts apparently were stronger, and he returned to Paris, France, where in 1885 he was graduated at the head of his class from the École des Ponts et Chaussées.

His first engineering engagement was with the late George S. Morison,[2] Past-President, Am. Soc. C. E., father of bridge building in America, and he continued with him in various capacities until August, 1893, when he opened an office in Chicago, Ill., as senior member of the firm of Modjeski and Nickerson. For a year they struggled with small bridge projects, surveys, and reports and finally dissolved the partnership. In 1894 Ralph Modjeski received his first major assignment, which was the design and construction of a 7-span double-deck railway and highway bridge over the Mississippi River at Rock Island, Ill. For the next few years he prepared standard designs for steel bridges, varying in spans from 10 ft to 250 ft, for the Northern Pacific Railway Company.

Although he had long been a friend of the late Alfred Noble,[3] Past-President, Am. Soc. C. E., it was not until 1902 that they collaborated on an entire project. While working under a partnership with Mr. Noble, Mr. Modjeski was appointed by the Southern Illinois and Missouri Bridge Company to be the chief engineer for the double-track railway bridge over the Mississippi River at Thebes, Ill. In the years that immediately followed many calls came for his expert service in the design and construction of railway bridges. In 1905 he was the chief engineer of reconstruction of the single-track railway bridge over the Missouri River at Bismarck, N. Dak., for the Northern Pacific Railway Company, and for the new double-track railway bridges in Portland, Ore., over the Columbia and Willamette rivers, the latter being known commonly as the Vancouver-Portland Bridges; and in 1906 for a new single-track electric railway bridge over the Illinois River at Peoria, Ill., for the Central Illinois Construction Company (Illinois Traction Company). His next work was the construction of the McKinley Bridge, a double-track railway and highway bridge over the Mississippi River at St. Louis, Mo., in 1907.

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[1] Memoir prepared by F. M. Masters, M. Am. Soc. C. E.

[2] For memoir, see *Transactions*, Am. Soc. C. E., Vol. LIV, June, 1905, p. 513.

[3] For memoir, *loc. cit.*, Vol. LXXIX (1915), p. 1352.

In 1908 he was appointed by the Government of the Dominion of Canada to the three-man Board of Engineers retained for the reconstruction of the Quebec Bridge over the St. Lawrence River. He served until the successful completion of this, the longest truss span in the world.

Between the years 1905 and 1915 Mr. Modjeski was the chief engineer on all bridges for the Oregon Trunk Railway Company between Celilo and Bend, Ore. These included the single-track railway bridge over the Columbia River at Celilo and the famous 340-ft, two-hinged arch that spans the Crooked River at a height of 350 ft above the stream. In 1910 he was engaged as chief engineer by the City of Portland to build the Broadway Bridge over the Willamette River. This was a double-track electric railway bridge, including a bascule span with roadways and sidewalks.

Plans for the Cherry Street Highway Bridge[4] over the Maumee River at Toledo, Ohio, had been prepared some time previous to 1912, but never executed. Finally the City of Toledo engaged Ralph Modjeski to redesign and construct the bridge. This was his first concrete arch bridge. In 1914 he designed and supervised construction of the Harahan Bridge, a double-track railway structure with wagon roadways over the Mississippi River at Memphis, Tenn. At the same time he did the reconstruction work on a double-deck, single-track railway and highway bridge over the Mississippi River at Keokuk, Iowa.

In 1917 Mr. Modjeski was consulting engineer in the preparation of the design for a double-track railway bridge over the Ohio River at Metropolis, Ill. In this connection he worked in conjunction with the late C. H. Cartlidge, M. Am. Soc. C. E., and after the latter's death was made chief engineer of construction. During that same period two projects for the New York, New Haven and Hartford Railroad Company were in hand: One a double-track railway bridge over the Thames River at New London, Conn., and the other the reinforcement and general reconstruction of the Poughkeepsie (N. Y.) Railway Bridge over the Hudson River.

In 1922 he was the engineer of design and consulting engineer of construction for the United States Government on the Tanana River Bridge in Alaska, a single-track railway bridge. From 1920 to 1924 he was also engaged in rebuilding two bridges under traffic: One was the double-track railway bridge with sidewalks over the Ohio River at Cincinnati, Ohio, which work was done without falsework, and the second was the double-track electric railway and highway bridge with sidewalks over the Missouri River at Omaha, Nebr. Following this work he was engaged as consulting engineer for the reinforcement of the cantilever span of the Columbia River Bridge at Wenatchee, Wash.

In 1923, in partnership with Frank M. Masters, M. Am. Soc. C. E., he designed and constructed two bridges over the Susquehanna River: A concrete arch highway bridge to replace the old covered timber bridge at Clark's Ferry, Pa., and a stone-faced arch highway bridge at Market Street in Harrisburg, Pa.

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[4] *Transactions*, Am. Soc. C. E., Vol. LXXX, December, 1916, p. 744.

In 1920 Ralph Modjeski had been selected by the Delaware River Joint Commission, together with the late George Smedley Webster,[5] Past-President, Am. Soc. C. E., and the late Laurence A. Ball, M. Am. Soc. C. E., as a member of the Board of Engineers to consider sites and prepare plans and estimates for the Delaware River Bridge. The report of the Board was submitted to the Committee the following year, and Mr. Modjeski was retained as chief engineer of the Commission, serving in this capacity during the entire period of construction until the bridge was opened to traffic. At the time of its completion in 1926, the Delaware River Bridge was the longest suspension bridge ever built, the length of the main span being 1,750 ft and the total length 9,570 ft. Upon the completion of this work, Clement E. Chase,[6] M. Am. Soc. C. E., who had been on Mr. Modjeski's staff from 1908 when he was first engaged as a rodman during his college holidays, became a member of the firm, and remained so until his death in 1933.

From 1927 to 1929 four bridges were built: A highway bridge over the Delaware River between Tacony, Pa., and Palmyra, N. J.; the Ambassador Bridge crossing the Detroit River between Detroit, Mich., and Sandwich, Ontario, Canada; a single-track railway bridge for the Texas and Pacific Railway Company over the Atchafalaya River at Melville, La.; and a cantilever highway bridge over the Ohio River between Louisville, Ky., and Jeffersonville, Ind. During this same period (1923 to 1930), Mr. Modjeski and his associates, in partnership with the late Daniel E. Moran,[7] M. Am. Soc. C. E. (famous substructure specialist of his time), designed and erected the Mid-Hudson Bridge at Poughkeepsie for the State of New York. This suspension highway bridge is particularly marked for its Gothic beauty.

The activities of the firm that immediately followed included two Henry Avenue bridges for the Department of Public Works, at Philadelphia, Pa.—one highway bridge over the Reading Railway tracks and one stone and concrete arch highway bridge over the Wissahickon Creek; a cantilever highway bridge, for the Indiana State Highway Commission, over the Ohio River at Evansville, Ind.; several structures for the Kentucky State Highway Commission, including the Highway Suspension Bridge over the Ohio River at Maysville (opened in 1931), the Smithland Bridge over the Cumberland River, and the Paducah Bridge over the Tennessee River. The next few engagements included the Huey P. Long Bridge over the Mississippi River at New Orleans, La., a double-track railway and highway bridge structure, plans for which were begun in 1926; plans and a report on the Portland Tualatin Tunnel at Portland, in 1929; two arches in Washington, D. C., in 1931—the Klingle Valley and Calvert Street bridges; studies and a report on the Baton Rouge (La.) Bridge over the Mississippi River in 1931; design and construction of the Owensboro (Ky.) and Cairo (Ill.) bridges over the Ohio River for the two respective Commissions; and the Davenport (Iowa) suspension bridge over the Mississippi River.

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[5] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 95 (1931), p. 1423.

[6] For memoir, *loc. cit.*, Vol. 99 (1934), p. 1430.

[7] For memoir, *loc. cit.*, Vol. 103 (1938), p. 1840.

It was in 1933, a few days after a contract had been signed for the design and construction of the rapid transit line and subways in connection with the Delaware River Bridge, that Mr. Chase suddenly met his death. It had been planned that he should care for the resident work of the project, so in this crisis one of Mr. Modjeski's old-time staff members, Montgomery B. Case, M. Am. Soc. C. E., was asked to fill in the gap. He became a member of the firm, thus making it Modjeski, Masters and Case. Mr. Case had worked with Mr. Modjeski consistently from 1906 with the exception of the period when he was on leave in the employ of the Port of New York Authority. Upon Mr. Case's retirement in 1938, and through the latter years of Mr. Modjeski's life, the firm continued work under the name of Modjeski and Masters.

Although Mr. Modjeski's professional life was noted for the erection of great structures, he was also a consulting engineer, and in that capacity was called in often on important works that never officially bore his name. Two such projects were for the City of New York: (1) The Manhattan Bridge in 1909, when he was engaged to report on the design and construction of the bridge and to supervise part of the construction, and (2) the proposed contract plans and specifications for major portions of the Triborough Bridge over the East River, on which he reported from 1930 to 1934. In 1916 he was appointed by the Public Service Corporation of New Jersey as a member of a commission of three to report on the feasibility of a vehicular tunnel under the Hudson River, including preliminary plans and estimates. In 1923 he was engaged by the Western Electric Company to check the plans and specifications for industrial buildings of their various plants, and in 1935 he reported to the Union of Socialistic Soviet Republics on the plans for the projected Palace of the Soviets at Moscow.

Reports on many of these bridges were published by Mr. Modjeski and filed with the Engineering Societies Library, in New York, N. Y., together with papers written on the technically unusual phases of some of his works. Among these articles is one which Mr. Modjeski read before the World Engineering Congress in Japan in 1929 at which he represented the engineers of the United States. Several others had been prepared for engineering societies and clubs of which he was a member.

Although no one person could properly appreciate all the aspects of this diversified personality, Ralph Modjeski's professional contributions have not lacked recognition. Of the many medals with which he was decorated, the outstanding are the Howard N. Potts Gold Medal in 1914, the Franklin Medals in 1922, the John Scott Medal in 1924, and the John Fritz Medal in 1930. In 1929 two honors came his way. He was awarded the Grand Prize by the Polish Government at the Exposition of Industry and Science at Posen, Poland, and he was selected as a representative of the United States at the World Engineering Congress in Japan. One decoration of which he was modestly but justly proud was that conferred in 1926 when the Republic of France made him a Knight of the Legion of Honor. Universities added honors "to one who had advanced human progress so greatly through his skill and devotion to the art and science of bridge building." Three times he received honorary degrees of Doctor of Engineering: In 1911 from the University of Illinois, at Urbana, Ill.; in 1927 from the Pennsylvania Military College, at Chester, Pa.; and in 1931 from the Polytechnic Institute, at Lwów, Poland.

In the latter year Mr. Modjeski was appointed Chairman of the Board of Consulting Engineers for the San Francisco-Oakland Bay Bridge, the longest major highway bridge in the world, which, with its approaches, extends 8¼ miles. It was in Mr. Modjeski's office and under his inspiration and guidance that most of the preliminary plans were drawn up, including the plan that was used for a central anchorage between the double suspension span for the west-bay crossing. This was the last major work on which Mr. Modjeski was active. Because of his failing health, he was forced to make his permanent home in California in 1936 so that he could be close to the work. Because of continued poor health, he was comparatively inactive for the last few years of his life. With his death the profession lost a pronounced personality and one who well merited the reputation of one of "the world's leading bridge engineers."

On December 28, 1885, Mr. Modjeski was married to Felicie Benda, of Cracow. They had three children: Felix Bozenta, Marylka Stuart (Mrs. Sidney Pattison), and Charles Emanuel John, who survive him. On July 7, 1931, he was married to Mrs. Mary T. Giblyn, who also survives him.

Mr. Modjeski was elected a Junior of the American Society of Civil Engineers on December 1, 1886; an Associate Member on July 1, 1891; and a Member on March 3, 1897.

DAVID ALBERT MOLITOR, M. Am. Soc. C. E.[1]

Died September 8, 1939

David Albert Molitor was born in Detroit, Mich., on August 16, 1866, the son of Edward Molitor and Catherine L. (Young) Molitor.

He received his early education in the Detroit public schools until 1881, at which time the family moved to St. Louis, Mo. After attending Smith Academy and St. Louis High School, he entered Washington University, in St. Louis, in 1883, but received the degree of Bachelor of Science in February, 1908, from George Washington University, in Washington, D. C. The following June he received the degree of Civil Engineer from the same institution.

During the summer of 1885, after completing his sophomore year, he gained his first practical experience under William T. Gould,[2] M. Am. Soc. C. E., on the Mobile and Ohio Railroad while the old St. Louis and Cairo narrow-gage road was being rebuilt.

In the summer of 1886 he studied photography and the various reproducing processes, built a lithographic printing press, and did about everything from portrait work to photolithographing. About 1904, this printing outfit was used by his father in developing Lake Survey chart printing in three colors, to be superseded later by a full lithographic printing establishment.

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[1] Memoir prepared by Clarence W. Hubbell, M. Am. Soc. C. E.

[2] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 104 (1939), p 1908.

In May, 1887, David sailed for Germany, expecting to remain there for some time. After spending three months in the office of his uncle, Jacob Schmitt, Oberingenieur of the Harbor and Highway District of Constance, Baden, acquiring a reading and writing knowledge of the German language, he secured a position with the Railway Department of the Grand Duchy of Baden and enjoyed the good fortune of participating in the design and construction of the strategical railway, Weizen-Immendingen, which was built to evade certain Swiss territory near Schaffhausen. This mountain railway, covering a distance of about 20 miles, presented very unusual difficulties. After spending one year on design, he was given charge of the construction of a 5-mile section of this double-track railroad. He was the official photographer and made several hundred 5-in. by 7-in. progress views. On completion of this work in 1890, he spent four months traveling through Germany, Switzerland, Bavaria, Austria, Hungary, Holland, and England, returning to his home town richer in knowledge of the world's works.

After returning to the United States, Mr. Molitor's first position was that of assistant engineer to the late Alfred Noble,[3] Past-President, Am. Soc. C. E., then resident engineer on the construction of the Mississippi River Bridge at Memphis, Tenn., with the late George S. Morison,[4] Past-President, Am. Soc. C. E., chief engineer. On completion of the river piers, which offered a valuable experience in deep pneumatic foundations, Mr. Molitor was put in charge of the Arkansas viaduct foundations. Shortly after the erection of superstructure had been commenced, he was placed in special charge of that class of work. As official photographer, he made daily 11-in. by 14-in. plates showing progress of erection.

In August, 1892, he became U. S. Assistant Engineer on River and Harbor Work at Detroit, spending a part of his time at Sault Ste. Marie, Mich. This work was under the late O. M. Poe, M. Am. Soc. C. E., General, Corps of Engineers, U. S. Army, until his death in October, 1895, and later under Col. G. J. Lydecker, Corps of Engineers. Joseph Ripley and the late Ebenezer S. Wheeler,[5] Members, Am. Soc. C. E., were in local charge at Sault Ste. Marie.

During the period from 1892 to 1898, Mr. Molitor had charge of important works and was entrusted with the preparation of specifications and superintending contract work connected with the Sault Ste. Marys Falls Canal locks; the 21-ft channel between Chicago, Ill., Duluth, Minn., and Buffalo, N. Y.; and many other improvements in the rivers and harbors of the Great Lakes, including extensive surveys and dredging operations.

Mr. Molitor also directed a party on precise spirit leveling from July, 1898, until July, 1899, for the U. S. Board of Engineers on Deep Waterways. This work extended along the St. Lawrence River from Cape Vincent to St. Regis, N. Y., and around Lake St. Clair, Michigan.

In December, 1899, he resigned his Detroit position, and also one he had accepted at Seattle, Wash., where he was to have had charge of the design and construction of the Lake Washington lock, under the late Harry Taylor,[6] M. Am. Soc. C. E., Captain (General), Corps of Engineers, and went to Fond du Lac, Wis., to become associated with his brother-in-law, Fred Rueping, in the conduct of a calfskin tannery.

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[3] For memoir, see *Transactions*, Am. Soc. C. E., Vol. LXXIX (1915), p. 1352.

[4] For memoir, *loc. cit.*, Vol. LIV, June, 1905, p. 513.

[5] For memoir, *loc. cit.*, Vol. LXXVI, December, 1913, p. 2249.

During his six years' residence in Fond du Lac, Mr. Molitor established a chemical and bacteriological laboratory for the Fred Rueping Leather Company, instituted a cost-keeping system, attended to plant alterations, and conducted research work for the control and improvement of tanning processes. He spent one semester at the University of Chicago studying bacteriology, histology, and anatomy.

Mr. Molitor also interested himself in public affairs, to better the sanitary conditions of his town. The storm sewage system, the pavements, and prospective river improvements—all resulted from lectures and newspaper articles in which he discussed the needs of a growing community, to which he freely gave the benefits of his thought and experience.

In 1906, when it was decided to build a lock canal across the Isthmus of Panama, on which his former friend, Joseph Ripley, was assistant chief engineer, the opportunity for returning to his chosen profession presented itself, and he became a designing engineer on locks and dams for the great canal. He visited the Isthmus in May, 1907, after the Canal Board was reorganized and the work was turned over to the Corps of Engineers. During the following year he had sole charge of the design for emergency dams.

When it was decided to move the designing force under the late H. F. Hodges,[7] M. Am. Soc. C. E., Colonel (General), Corps of Engineers, from Washington to the Isthmus, and the designs for the emergency dams were practically completed, Mr. Molitor resigned his position to accept a professorship at Cornell University, in Ithaca, N. Y., in charge of the Department of Topographic and Geodetic Engineering, a position which he held from 1908 to 1911.

He spent the year, 1911–1912, in Kansas City, Mo., as designing engineer with the firm of Waddell and Harrington, and was engaged in the design of numerous large vertical lift bridges. He also prepared designs for the lower lock gates of the Keokuk lock for Hugh L. Cooper,[8] M. Am. Soc. C. E., in 1912, and for several months was occupied with special work for the New York Barge Canal.

From February, 1913, to May, 1916, Mr. Molitor was chief designing engineer for the Toronto (Ontario, Canada) Harbor Commissioners and during that period made designs for several large lift bridges, harbor head-walls, sea-walls, and numerous other structures, finally conducting some observations on wave pressures which led to a professional paper on this subject.

In the spring of 1916, Mr. Molitor received a call from Carl E. Schmidt to join him in his business, a calfskin tannery in Detroit. With the prospect of long delay in the prosecution of work in Canada, during and after the war, this proposition was attractive, and so he accepted the call and became construction, efficiency, and chemical engineer, thus returning for a second time to the tanning industry.

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[6] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 95 (1931), p. 1609.

[7] For memoir, *loc. cit.*, Vol. 94 (1930), p. 1528.

[8] For memoir, *loc. cit.*, Vol. 103 (1938), p. 1772.

In November, 1923, Mr. Molitor again returned to engineering in the structural department of Albert Kahn, Inc., with J. T. N. Hoyt, M. Am. Soc. C. E., chief engineer. In this department, for eight years, he was engaged in checking designs of industrial and office buildings and later in designing all types of construction in steel and reinforced concrete.

In 1932, Mr. Molitor was reinstated in the U. S. Government service, in the Procurement Division of the Treasury Department, as government representative on post offices. This work in Michigan included the post offices at Bay City, Lansing, and Lapeer. In July, 1934, he was transferred to the head office in Washington, D. C., and put in charge of surveys of public buildings, where he remained until his retirement in December, 1937.

In June, 1938, Mr. Molitor moved to Harlingen, Tex., where he died September 8, 1939, in the seventy-third year of his life.

For more than forty years, Mr. Molitor made frequent contributions to the engineering literature of his day, covering a wide range of technical subjects. He was also the author of the following treatises: "Hydraulics of Rivers," 1908, John Wiley and Sons; "Kinetic Theory of Engineering Structures," 1911, McGraw-Hill Book Company; "Least Square Adjustments," 1916 (manuscript only); "Practical Chimney Design," 1937, and "Structural Engineering Problems," 1937, both published privately, Washington, D. C.

Mr. Molitor was married twice, his first union being with Clara K. Rueping of Fond du Lac, to whom he was married on June 28, 1893. To them were born three daughters: Margaret, Clara, and Anita, all of whom are married and survive him. His second wife, to whom he was married in January, 1914, was Mabel H. White, of Toronto, who survives him.

Socially, he was always worthy of the high appreciation of his many friends and acquaintances. His artistic sense, so clearly manifested in his works and writings, was characteristic of his general make-up. He was very active in music, played the flute in the theater orchestra of Fond du Lac, and during one winter conducted a thirty-piece orchestra giving Sunday afternoon concerts. He was an ardent lover of Nature and things beautiful. His achievements in chemistry, microscopy, bacteriology, and photography deserve special mention. In politics, he was non-partisan, and in spiritual matters he was an agnostic.

In January, 1907, he was appointed a member of a special committee of the Society on the status of the metric system, and submitted a report at the Annual Meeting, in 1908. In 1921 he served as President of the Detroit Section of the Society. He was also a member of the Engineering Institute of Canada, and a charter member of the Detroit Engineering Society. Fraternally, he was identified with the Masons, Knights of Pythias, and Benevolent Protective Order of Elks.

On February 22, 1932, the Board of Trustees and Faculties of the George Washington University conferred on Mr. Molitor the degree of Doctor of

Engineering, *honoris causa,* with the citation of President C. H. Marvin—"Partner in Governmental undertakings, builder of standards in engineering, a sincere man."

On August 3, 1939, he was honored by a Certificate of Life Membership by the Engineer Alumni Association of the George Washington University.

Mr. Molitor was elected a Member of the American Society of Civil Engineers on October 6, 1897.

ARNALDO ALVES da MOTTA, M. Am. Soc. C. E.[1]

Died October 1, 1938

Arnaldo A. da Motta was born in the City of Belém, the capital of Pará State, Brazil, on December 2, 1895, the son of Alberto Alves da Motta and Cecilia Ribeiro da Motta.

In 1903, he began his studies at the Liceu Figueirense, in Figueira da Foz, Portugal, and he continued them in 1905 at the Grünau Institute, in Bern, Switzerland. He was the youngest pupil at the latter school, and attracted the attention of every one by his characteristic vivacity.

He went to England in 1910 to attend Cheltenham University, at Cheltenham, and in 1912 he entered the Technology School in Manchester, England, where he began his engineering studies. Because of the World War, the school was partly closed in 1914, and Mr. Motta returned to Brazil in 1915, where he entered the Mackenzie College in the State of São Paulo. He received the degree of Civil Engineer in 1922.

If, during his academic years, Arnaldo Motta's ability had been outstanding, it was even more so in his professional undertakings, which, in their various branches, were always successful.

Before the conclusion of his course he took the position of technician of the Associação Paulista de Estradas de Rodagem on the location and construction of highways, working from January to December, 1921. About this time he formed a partnership with J. Paiva Meira, and, under the firm name of Paiva e Motta, they completed, for the São Paulo State Government, 43 km (26.8 miles) of highway between the cities of São Paulo and Ribeirão Preto. This included 6 reinforced concrete bridges and 18 km (11.2 miles) of highway between Pindamonhangaba and Rosario, also in São Paulo State. The partnership was dissolved in December, 1923, by mutual consent.

Arnaldo Motta was employed as a government engineer in the Departmento do Serviço Sanitario, supervising the location and construction of the sewerage plants. At the same time he directed the work of his own firm and opened a branch office in Rio de Janeiro, Brazil, in partnership with J. J. Barbosa. This office was closed in 1930.

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[1] Memoir prepared by Henrique Pegado, M. Am. Soc. C. E.

This was a period of great activity, and his work amounted to the sum of $40,005,000 at the exchange rate of that period. In June, 1925, he contracted for and rebuilt the railway station in Fortuna, São Paulo State; and constructed a large coffee warehouse, for the Sorocabana Railway, São Paulo. From July, 1924, to December, 1927, he contracted for and constructed the following highways for the São Paulo government, being in responsible charge of grade alinement, design, drainage, bridges, etc.: 5.8 km (3.6 miles) from Espirito Santo do Pinhal to São João da Bôa Vista; and 5.8 km from Espirito Santo do Pinhal to Jardim, all communities on the main highway between the States of São Paulo and Minas Geraes; 6.0 km (3.7 miles) from São José dos Campos to Formosa on the main highway between São Paulo and Rio de Janeiro; 5.2 km (3.2 miles) from Formosa to Lambary; and 6.0 km from Bananal to Pouso Secco.

Between the communities of Gramadinho and Jardim, he built 7.8 km (4.8 miles) of highway between São Paulo and Paraná states and a concrete bridge consisting of two spans 24 m (78.7 ft) long. During this time he had charge of surveys and supervised the plans and design for a highway 74.8 km (46.4 miles) long between Santo Amaro and Itanhaem, and another 28-km (17.4 miles) highway from Caçapava to Jambeiro, both in the State of São Paulo. In December, 1925, he built 12 km (7.4 miles) of highway with four bridges for the Comissão Obras Novas de Rio Claro, São Paulo State. From June, 1926, to June, 1927, he constructed the fruit market and garage for the City of São Paulo, São Paulo. He designed and constructed six private houses, from 1927 to 1930. He designed a country house for himself and his family on Santo Amaro Lake, São Paulo, in the Portuguese-Spanish mission style, which he built in 1931.

From January, 1927, to 1929, he contracted for, designed, and constructed highways for the federal government including 70 km (43.3 miles) of waterbound macadam from Taquara to Guaratiba in the Federal District; 45 km (27.8 miles) from Vargem Grande to Bandeirantes; and 20 km (12.5 miles) of concrete pavement on the automobile Circuit of Gavea. He also designed and constructed the beautiful Joá Restaurant, on Gavea Highway, overlooking the sea—a spot greatly renowned to tourists visiting Rio de Janeiro.

From Irajá to Madureira, also in the Federal District, he relocated the principal street named Monsenhor Felix, laid sidewalks, reorganized the sewerage system, and paved the street with bituminous concrete—thus thoroughly transforming the appearance of this sleepy little town. He located and constructed several kilometers of highway between Irajá and Vigario Geral; designed and constructed a crushing plant, and installed a rotating crusher for the municipality of the Federal District.

He laid 10 km (6.2 miles) of concrete pavement on the mountainous highway between Petrópolis and Rio de Janeiro, one of the most beautiful highways of the world, with its serpentine curves, amid dense and tropical vegetation, overlooking the wonderful Rio Bay. From 1928 to 1930 he also constructed several kilometers of railroad for Sorocabana Railway, São Paulo State, in a very mountainous zone, from Mayrink to Santos.

In August, 1930, he was appointed Brazilian Delegate to the Sixth International Road Congress, held in Washington, D. C. He attended the congress, saw many improvements in road-building technique, and met several distinguished American engineers with whom he frequently corresponded. He always used to say that the engineering future lay in America and that if his son should become an engineer, he would like to have him take a finishing course in the United States.

From January to April, 1931, he traveled in France and Belgium where he visited some coal mines. He was particularly interested in mining as he had been invited by the Compannia Ribeirão Novo to take charge of a coal mine in Paraná. He operated the mine from April, 1931, until his death. He was appointed manager of the company in 1932. During his administration he reorganized the coal-mining plan, installed new machinery, and constructed new houses for the miners.

These professional activities, which, as can be noted, represent a great career, were all confined to a relatively short period from 1921 to 1931. After these full years, he slowed down his activities and dedicated the greater part of his time to study and to publishing some interesting engineering data. Nevertheless, he still continued his professional work. He promoted a master plan for Chacara do Carvalho, São Paulo City, São Paulo, with complete platting and detailed estimates of costs of earth excavations, brick pavements, concrete sidewalks, curbs, water works, sewerage, and gas mains. He constructed an 8-km (5-mile) highway between the communities of Piedade and Juquiá in São Paulo State.

In October, 1933, he was appointed delegate of S. Paulo Automobile Club and Municipality of São Jõao da Bôa Vista, São Paulo State, to attend the Fifth National Road Congress, held in Rio de Janeiro.

In August, 1934, he was appointed Brazilian Delegate to the Seventh International Road Congress held in Munich, Germany. He attended the Congress and viewed nearly all European highways in order to appreciate and see their technique. He traveled in his own car, with his wife, all over Germany, Austria, Hungary, Switzerland, Italy, France, and part of England. On his return he published a book entitled "European Highways," in which he related his impressions on the various highway construction methods and an account of the proceedings of the Congress.

Arnaldo Motta always demonstrated a great interest in highways, and in his early years, even before this problem had attracted much attention among Brazilian engineers, he saw with keen foresight the important rôle to which they were assigned in the development and future of such a large country as Brazil. He chose for his engineering thesis, "Records and Studies on Highways of the Northern Sea-Coast of the State and Types of Economical Drainage," in which he showed a great comprehension of this branch of engineering.

In 1934 he published data on public contracting for highway construction, in which he explained and recommended that the use of standard proposals should be adopted by government departments, in judging public proposals of the qualified contractors. This work was appreciated by the public authorities and adopted by the Road Department of the São Paulo State. It was because of the many discrepancies occurring in the qualifying proposals (which Arnaldo Motta had occasion to verify) that he decided to write this useful booklet.

In 1936, during the Sixth National Highway Congress in Rio de Janeiro, Mr. Motta presented a thesis on "Low-Cost Paving for Highways of Small Traffic," emphasizing two important factors—economy in time and money, and up-to-date surfacing. Highways present a problem in a country as large as Brazil, with its vast unexplored areas and its earth roads—for earth roads are not practical today, and high-cost paving is not possible. So, he recommended a grade stabilization of the roads with chloride of calcium or sodium, which not only insures a perfect surface but also suppresses dust and mud. Arnaldo Motta was complimented by the Congress chairman for his valuable suggestions.

In 1935 he was asked by the São Paulo State Government to present a plan for remodeling the state highway between São Paulo City and Santos, the important port of São Paulo State. He made an elaborate and economical study for that highway, on the so-called basis of self-financing—that is, toll to be charged on the highway until the highway had paid for itself. Thereafter it is free. This plan has been adopted in several countries.

During 1937 and 1938, Mr. Motta had acquired new and important modern machinery for road building. He began using this machinery on road construction in the community of Santo Amaro, São Paulo State, which work he did not live to finish. At the same time he was finishing a highway from the communities of Joanopolis to Piracaia, São Paulo State. Both of these works were completed after his death by his firm under the supervision of his brother Alberto Alves da Motta, and an engineer.

Mr. Motta was an honorary member of the Instituto de Engenharia of São Paulo, and the Automobile Club of Brazil; a life member of the "Association Permanente des Congrés de la Route"; and a member of the Rio de Janeiro Club de Engenharia, and the American Road Builder's Association, Washington, D. C. He was also affiliated with the following private clubs: São Paulo Automovel Club, Fluminense Yacht Club, Touring Club, Club Athletico Paulistano, Sociedade Harmonia de Tennis, Yacht Club Paulista, São Paulo Country Club, and Hippica Pulista.

He was married on January 24, 1924, to Heloisa Alves de Lima, daughter of the late Prof. João Alves de Lima, eminent surgeon and member to the American Fellowship of Surgeons, and D. Elisa de Barros Alves de Lima, a daughter of the Barões de Piracicaba. Arnaldo Motta was a model husband and father. He left five children, Heloisa-Lourdes, Maria-Lisah, Arnaldo-Marcos, Carmen-Silvia, and João-Eduardo. He was a companion to his family, whom he cherished fondly. His death is felt deeply, not only by his kin, but by his large circle of friends.

He was killed in a tragic airplane crash on October 1, 1938, caused by unfavorable atmospheric conditions. He was piloting his own plane and accompanying the São Paulo State Governor on a trip to the interior of the state. State honors were presented to him at his funeral.

Arnaldo Motta was elected an Associate Member of the American Society of Civil Engineers on May 25, 1931, and a Member on October 14, 1935.

JOHN WILLIAM MUSHAM, M. Am. Soc. C. E.[1]

Died December 7, 1935

John William Musham was born on November 10, 1877, in Chicago, Ill., and died of a cerebral hemorrhage on December 7, 1935, at his home in Oak Park, Ill. He was the son of William H. Musham and Catherine (McFadden) Musham. His ancestors, on his father's side, came to America from a district in Northern England near the Scottish border in the County of Northumberland. His grandfather, William L. Musham, settled in Chicago in 1835 and died in that city in 1844. His maternal ancestors came from Westport in Northwest Ireland, in the province of Connaught. John's father was a member of an early Chicago Volunteer Fire Company and helped fight the great Chicago fire of 1871. He was also a veteran of the Chicago Fire Department, and chief marshall of that department from 1901 to 1904.

Mr. Musham was educated in the Chicago public schools; LaSalle Institute, Armour Institute of Technology (later the Illinois Institute of Technology) both in Chicago; and the University of Illinois, in Urbana, Ill. He was graduated from the latter in June, 1898, with the degree of Bachelor of Science in Civil Engineering. In addition to his technical education and his continued professional study, he had a deep interest in general literature and was a student of history and political science.

Immediately after graduation, he returned to Chicago where he was employed by Edward Gleason and Sons, Contractors, and Holabird and Roche, Architects. In 1899 he entered the service of the Bridge Department of the Chicago, Milwaukee and St. Paul Railway Company, as designer and detailer under the late Onward Bates,[2] Past-President and Hon. M. Am. Soc. C. E., then engineer and superintendent of bridges and buildings. Mr. Musham remained there until January, 1902. During the latter part of this period, the late Charles F. Loweth,[3] Past-President, Am. Soc. C. E., had succeeded Mr. Bates as engineer and superintendent of bridges and buildings.

In 1902 and the early part of 1903, he was employed as checker and squad master in the American Bridge Company's office. In the latter part of 1903 he became designing engineer of the steel work for the new third-track construction for the Chicago and South Side Elevated Railway Company. In 1904, 1905, and the early part of 1906 he designed two counterbalanced swing bridges for the Sanitary District of Chicago and made the general design for an emergency dam at Lockport, Ill. In the latter part of 1906, he became estimator and engineer for the Bates and Rogers Construction Company, in Chicago, and for the remainder of that year and a part of 1907 he superintended the building of a reinforced-concrete warehouse for Holabird and Roche, Architects. In the early part of 1908 he checked the design of the movable leaf and operating machinery for the Polk Street Bascule Bridge for the Strauss Bascule Bridge Company, in Chicago, and the latter part of that year was superintendent of construction of the reinforced-concrete Moos Building for S. N. Crowen, architect.

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[1] Memoir prepared by I. F. Stern, T. L. Condron, and the late Joseph H. Prior, Members, Am. Soc. C. E.

[2] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 102 (1937), p. 1484.

[3] For memoir, *loc. cit.*, Vol. 101 (1936), p. 1488.

From January, 1909, to May, 1925, Mr. Musham was associated as computer and designing engineer with Theodore L. Condron, the late Frank F. Sinks,[4] and Chester L. Post, Members, Am. Soc. C. E., first in the firm of Condron and Sinks, and later in the organization of the Condron Company, Consulting and Designing Engineers, except during 1917 and 1918.

During the period of World War I from June, 1917, to July, 1919, he was commissioned as Captain of Engineers at Camp Sam Houston, Texas, and served with distinction in the American Expeditionary Forces in France.

While associated with Condron and Sinks and the Condron Company, Mr. Musham was in charge of the computations and designs of many structures, including the structural steel for the freight house for the Missouri, Kansas and Texas Railway Company. This 400-ft long, multi-story building involved the unusual column spacing of 38 ft by 40 ft and a first story height of approximately 40 ft. At this time these panel dimensions probably exceeded those of any other multi-story building in the United States.

He had an important part in the development of the reinforced-concrete building designs for which the Condron Company became noted. In 1923 he was in charge of the computations for the design of the eighteen steel continuous girders of three spans each, over four supports, ranging in over-all lengths from 160 ft to 300 ft for the South Park Boulevard Viaduct at 23d Street over the right of way of the Illinois Central Railroad Company in Chicago. At the time, this was a unique structure, as the supporting steel columns, as well as the girders, were designed for bending stresses resulting from loads and temperature variations. This design was computed and designed before the development and publication of the present extensive literature on "rigid-frame structures." Mr. Musham's elaborate analysis of the stresses in this structure, unfortunately, never was put in shape for publication, but the successful completion of this viaduct led to the building of similar structures throughout the United States. Mr. Musham was an engineer of unusual talent as a computer and designer.

In May, 1925, he withdrew from the Condron Company to become general superintendent of the Mid-Continent Construction Company, and later chief engineer for the Herlihy Mid-Continent Company, which position he held at the time of his death. Some of the more important contracts on which Mr. Musham was engaged include the two-level structure for Wacker Drive along the south side of the Chicago River, in Chicago. The upper level of this structure is a reinforced-concrete flat slab 120 ft wide by 5,000 ft long and is supported by reinforced-concrete columns carried on reinforced-concrete cylinders (caissons) extending down to bedrock. The contract cost of this improvement was approximately $7,000,000. The Southwest Sewage Disposal Plant and the Calumet Sewage Treatment Works for the City of Chicago, the

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[4] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 104 (1939), p. 2002.

State-Line and Michigan City Generating Stations, the foundations for the Chicago Board of Trade, and the Palmolive Building and other large buildings were constructed by this company while Mr. Musham was chief engineer.

He was appointed a member of the first Illinois Board of Examiners of Structural Engineers by the late Governor Edward Dunne in 1916. He served on that Board until 1919 when the new Department of Registration and Education was created by a law that abolished separate Boards of Examiners for architects, engineers, and other professions and trades. He was a member of the Western Society of Engineers, the Oak Park Post No. 15 of the American Legion, and St. Catherine's Roman Catholic Church of Oak Park.

Mr. Musham is survived by his widow, Ella McNellis Musham, to whom he was married on October 14, 1914. A son, William C.; a daughter, Jeanne; a brother, Major Harry A. Musham, U.S.A. (retired); and two sisters, Anna and Ella Musham, also survive him.

Mr. Musham was elected a Member of the American Society of Civil Engineers on September 2, 1914.

HERMAN CHRISTOPHER NEUFFER, M. Am. Soc. C. E.[1]

DIED AUGUST 23, 1940

Herman Christopher Neuffer was born on October 12, 1891, at Preston, Idaho, the son of John and Louisa (Zollinger) Neuffer. He received his early education in the grade schools and at Oneida Academy, in Preston. He entered the University of Idaho at Moscow, Idaho, in 1911, and was graduated with the degree of Bachelor of Science in Civil Engineering in June, 1915.

Shortly after graduation, Mr. Neuffer was employed as chief of party on irrigation, canal, and highway surveys for Franklin County, Idaho; in January, 1917, he was made county engineer and continued in this capacity until May, 1917, when he resigned to accept a Civil Service appointment as assistant engineer with the U. S. Indian Irrigation Service in Albuquerque, N. Mex.

With the coming of the World War, Mr. Neuffer resigned his appointment on December 27, 1917, and entered the Third Officers' Training Camp at Fort Riley, Kans. He was commissioned as a Second Lieutenant and assigned to the 89th Division, Camp Funston, Kansas. He received his honorable discharge from the Army at that camp on February 3, 1919.

Immediately following his discharge, and until July of that year, he was employed as a civilian engineer with the War Department at Camp Henry Knox, Kentucky. During this time he served as chief of party on highway location and construction. With the suspension of this work, he made application and was reappointed, in July, 1919, to his previous position as assistant engineer in the Irrigation Division of the U. S. Indian Service at Albuquerque.

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[1] Memoir prepared by a Committee of the New Mexico Section composed of W. A. Laflin, Assoc. M. Am. Soc. C. E., and Maurice C. Lipp, Jun. Am. Soc. C. E.

He remained with the Indian Service until May, 1922, when he was appointed chief engineer with the Palo Verde Joint Levee District, Palo Verde Drainage District, and Palo Verde Mutual Water Company, which covered the Palo Verde Valley in California, the headquarters being located in Blythe, Calif. This project consisted of levee and flood protection construction, and also drainage and irrigation on a 70,000-acre project.

In May, 1923, Mr. Neuffer entered the employ of M. H. Whittier Company of Los Angeles, Calif., to study and report on the topography and ground water on their property in Shasta County, California. With the completion of this report in September, 1923, he entered private practice in El Monte, Calif., devoting most of his time to irrigation studies and development.

In February, 1924, he discontinued his private practice and re-entered the U. S. Indian Irrigation Service in the Los Angeles office, where he was engaged upon the design and construction of the Florence-Casa Grande Project in Arizona under the supervision of Herbert V. Clotts, M. Am. Soc. C. E.

In April, 1925, he was transferred to the supervision of the late Charles R. Olberg,[2] M. Am. Soc. C. E., assistant chief engineer in charge of the design and construction of Coolidge Dam in Arizona. In this design work and investigation, there was evolved a new type of dam known as the "multiple dome." This design was passed upon by an eminent board of consulting engineers and adopted by the government. Mr. Neuffer's connection with this work was fittingly described by his supervisor, Major Olberg, when he wrote: "The successful consummation of the design of the Coolidge Dam is largely due to the loyal and indefatigable work of Mr. Neuffer."

With the award of the contract for construction of the dam, Mr. Neuffer was placed in charge of the office engineering and laboratory testing at the dam site near San Carlos, Ariz. In addition to these duties he assisted the government counsel in preparing the Gila River water adjudication suit.

With the completion of Coolidge Dam in 1929, Mr. Neuffer was appointed engineer at large in charge of Indian Service interests in the Middle Rio Grande Conservancy District project at Albuquerque. On July 30, 1930, he succeeded Supervising Engineer H. F. Robinson, who retired. He assumed this appointment while continuing his connection with the Middle Rio Grande project. Mr. Neuffer remained as supervising engineer in charge of irrigation on Indian lands in Arizona, New Mexico, Colorado, and Oklahoma until October, 1936, at which time he formed a consulting partnership with W. R. Holway, M. Am. Soc. C. E., of Tulsa, Okla.

In the following years, until his death, Mr. Neuffer was employed as a consultant on numerous important projects. His work included irrigation and hydroelectric projects, and also the design of sewage disposal plants. His firm had charge of the design and plans for the Grand River hydroelectric project in Oklahoma.

Mr. Neuffer maintained a keen interest in civic affairs and was regarded as one of the most active and energetic men in the state. He served as National American Legion executive committeeman from New Mexico and was past-commander of the State Legion department. He was serving his second term on the Community Chest Board of Directors, having been elected Chest President in 1939. He was a member of the Unemployment Compensation Commission for New Mexico by appointment from the Governor in 1939. Mr. Neuffer was an honorary member of Sigma Tau engineering fraternity, and also was affiliated with the Temple Masonic Lodge, Ballud Abyad Shrine, and the Methodist Church.

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[2] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 105 (1940), p. 1896.

Mr. Neuffer was keenly interested in the activities of the Society. He was a leader in the group which organized the New Mexico Section, later serving a term as President of the Section.

Surviving Mr. Neuffer are his wife, Virginia (Simmons) Neuffer, to whom he was married on April 2, 1921; a son, Bruce; a daughter, Virginia; his parents; six brothers; and three sisters.

Funeral services and burial were conducted in Albuquerque on August 26, 1940.

Mr. Neuffer was elected a Member of the American Society of Civil Engineers on April 7, 1930.

WALTER SWAIN NICHOLS, M. Am. Soc. C. E.[1]

Died March 10, 1940

Walter Swain Nichols was born in Germantown, Philadelphia, Pa., on June 28, 1869, the son of George and Martha (Swain) Nichols. He attended Germantown Academy, but did not go to college because of the death of his father. After leaving the Academy, he attended the School of Industrial Art, at Philadelphia. In 1890 he secured a position with the Norfolk and Western Railway Company, this being his first work of an engineering character. He occupied a number of positions with the railroad, as follows: Chainman, rodman, instrumentman, assistant supervisor, and acting supervisor on the construction of the Ohio extension.

He then went into the drawing room of the Maintenance of Way Department. Later he was put on construction work, being assistant engineer of construction on the railroad's Radford cutoff. In this position he was in charge, under the engineer of construction, of heavy grading, culverts, retaining walls, cofferdams, bridge masonry, and a 3,300-ft tunnel. The work with the Norfolk and Western Railway Company covered a period of ten years.

After leaving the railroad, Mr. Nichols did engineering work for a number of companies, as follows: The Union Traction Company of Philadelphia, on track construction, repairs, and alterations to track; the Pennsylvania Railroad in charge of party on construction of their lines to the mines above

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[1] Memoir prepared by Thomas Earle, M. Am. Soc. C. E.

Windber, Pa.; and Ford, Bacon and Davis of New York, N. Y., in charge of design and construction of 4 miles of track for the Ardmore and Llanerch Street railway companies.

Mr. Nichols next went to Dallas, Tex., for Johnston Elliott and Company. Here his work consisted of making a valuation of the old lumber railroad of the Jefferson and Northwestern Railway Company, and plans and estimates for a 6-mile extension, which was built later.

At this time, when Mr. Nichols was about forty years old, his fondness for artistic work led him to branch out into the making of surveys and maps for the construction of landscape work. This included the following: Location map, topographical survey, and estimates for a railroad between Wyomissing and Ephrata, Pa., 17.8 miles long; plans and construction of landscape work for large estates, drives, grading, roads, etc.; and topographical surveys and small mine survey for the Lost River Tin Mining Company, on Lost River west of Teller, Alaska, to accompany a report of F. C. Fearing, geologist, of Bronxville, N. Y.

Between 1904 and 1912 Mr. Nichols was engaged for three or four periods of several months each by the Philadelphia geologist and mining engineer, Edward V. d'Invilliers, to conduct extensive stadia surveys to determine locations and elevations of coal openings and diamond-drill holes made to exhibit the coal resources of the areas under examination. Most of this work was done in eastern Kentucky in what are now the prominent Elkhorn and Harlan coal-mining districts. This territory is very mountainous, and the work required a considerable degree of skill. Mr. Nichols made similar surveys for Mr. d'Invilliers in the coal fields of central Pennsylvania.

He was superintendent on construction of the golf course of the Burning Tree Country Club, at Bethesda, Md., and made the plans and surveys for other golf courses, all for Toomey and Flynn. During the absence of Reginald H. Patterson, Assoc. M. Am. Soc. C. E., he acted as consulting engineer for White Sulphur Springs, Inc. He also designed and superintended construction of a small earth dam with concrete core wall (6-ft head at Blue Bell, Pa.). His topographic maps were noted for their accuracy and beauty. He was very facile with his hands and made an elaborate relief model of the Cascade Golf Course of the Virginia Hot Springs Company. While engaged on these various pieces of work, Mr. Nichols also made a number of beautiful landscape paintings.

During the last five years of his life he was in poor health and did not do much work. He had an affable and agreeable disposition and his friends and associates were very fond of him. He was interested in all matters relating to federal and civil activities.

He was married on February 4, 1922, to Emelyn Slaughter, who survives him.

Mr. Nichols was elected a Member of the American Society of Civil Engineers on February 4, 1913.

THEODOR SMIDT OXHOLM, M. Am. Soc. C. E.[1]

DIED JULY 1, 1940

Theodor Smidt Oxholm was born at Willets Point, Long Island, N. Y., on June 29, 1866. He was the son of Carl W. E. Oxholm and Susan L. (Kellogg) Oxholm. His father was a Danish engineer, who came to New York in 1852 and shortly thereafter became connected with the United States Military Service as construction engineer for Fort Totten, at Willets Point.

Mr. Oxholm received his preparatory education at St. Paul's School, in Garden City, N. Y., and at the Flushing (N. Y.) Institute, after which he entered Cooper Union, in New York, N. Y. He was graduated in 1889 with the degree of Bachelor of Science. Later he received the degree of Civil Engineer from the same institution.

On July 2, 1884, he was appointed rodman in the Department of Public Works, New York City, the appointment having been made from what is believed to be the first city eligible list promulgated as the result of competitive examination.

His service for the City of New York included assignments to the Quaker Bridge Dam and other reservoir projects in the Croton Water Shed, and responsible charge of paving work in the city; in the meantime he was promoted, successively, through the positions of leveler and transitman.

His city service was interrupted on two occasions by employment on the construction of the Spuyten Duyvil ship canal and supervision of paving work in the City of Yonkers, N. Y.

In January, 1894, Mr. Oxholm left the service of the City of New York to become chief engineer of the Village of New Brighton, Staten Island; but, with the consolidation of 1898, under which Staten Island became a part of "Greater New York," his status as a New York City engineer was restored.

From that time until his retirement on March 21, 1933, he devoted himself to the upbuilding of Staten Island, advancing in engineering responsibility from senior engineer through the higher grades and to the position of consulting engineer, to which he was appointed on May 1, 1926.

To his energy and skill is due the planning and construction of the sewerage, highway, and park systems of the Borough of Richmond. He contributed in no small degree to the great municipal improvements which were executed through his incumbency of office, in which period the Borough made great headway in changing from a group of small villages into an important part of a great metropolis. During his entire service and under changing administrations, he did his work quietly, but forcefully, winning the respect and confidence of his superiors as well as of the citizens of Staten Island as a whole.

In his younger days he was greatly interested in canoeing. He won the International Sailing Canoe Race off Coney Island in 1892 and the International Challenge Cup in 1912. For these achievements he was elected an Honorary Member of the American Canoe Association.

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[1] Memoir prepared by Arthur S. Tuttle, Past-President and Hon. M. Am. Soc. C. E.

He was an active member of the Municipal Engineers of the City of New York and was honored with the office of President.

On his retirement from city service, he gave up the active practice of his profession. He devoted the succeeding years to various hobbies with continued interest in the affairs of Christ Church, New Brighton, of which he was a vestryman, and to the betterment of his community.

On October 3, 1894, he was married to Louise BeVier Deyo, who survives him. Also surviving are three sons—Carl, Philip D., and Theodor—and one daughter, Catharine Bevier (Mrs. Northrope Jones).

Mr. Oxholm was elected a Member of the American Society of Civil Engineers on November 3, 1897.

JAMES EMERSON PHILLIPS, M. Am. Soc. C. E.[1]

DIED OCTOBER 26, 1940

James Emerson Phillips was born in Russellville, Ind., on July 8, 1883, the son of Eli Andrew and Evaline (Odell) Phillips. He received his early education in the public schools in Indiana, later attending Bloomingdale Academy, in Bloomingdale, Ind., and Wabash College, in Crawfordsville, Ind., during the years 1898 to 1902. His technical training was obtained at Rose Polytechnic Institute, in Terre Haute, Ind., which he entered in 1903.

From March to August, 1905, he acted as rodman with the Chicago and Eastern Illinois Railroad Company. In July, 1906, he secured a position with the Chicago and North Western Railway Company, in which he served as rodman, instrumentman, draftsman, computer, and inspector. Near the end of the engagement he acted as inspector on pile-trestle and wood-truss bridge construction in South Dakota.

Between April and September, 1907, Mr. Phillips conducted a complete topographic survey of a large country estate near Cleveland, Ohio. Commencing in 1908, and continuing during 1911 and 1912, he engaged in private civil engineering practice at Rockville, Ind., and Portland, Ore. Between periods of private practice he served as county surveyor for Parke County, Rockville, during 1909 and 1910. In this position he was in charge of surveys, designs, and specifications for, and was superintendent in charge of, construction of 40 miles of highway, including numerous reinforced-concrete bridges.

In answer to the call of the West, Mr. Phillips sought larger fields in the growing communities on the Pacific Coast. He again entered private engineering practice at Portland in 1911, continuing until 1912. In June of the latter year he became associated with the Bureau of Water Works and Supply of the City of Los Angeles, Calif., where he was engaged in an executive capacity as engineer in charge of the Los Angeles Aqueduct at the time of his death.

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[1] Memoir prepared by Burton S. Grant, Assoc. M. Am. Soc. C. E.

Mr. Phillips possessed a keen desire for the outdoors and particularly the country traversed by the Los Angeles Aqueduct. He often mentioned to his colleagues in the Los Angeles Water Department his early desire to be associated with the aqueduct of which he had read newspaper accounts while still in the East. His first project as a draftsman when entering the Water Department was designing the Cascades at the southerly terminus of the aqueduct where Owens Valley water was aerated before entering the distribution system. Although subsequent assignments took Mr. Phillips into the field of design and construction of major water supply lines and extensive distribution systems in the San Fernando Valley, he still had an unusual interest in the aqueduct system. In the office as draftsman, and later as chief draftsman, to which position he was appointed in 1919, he conceived and prepared many records of the aqueduct and its features which have greatly facilitated its maintenance and operation during the succeeding twenty-eight years. Among these was a condensed profile of the system, as constructed, which profile is still used constantly in connection with aqueduct operation and maintenance.

In 1924 he was promoted to the position of assistant civil engineer having charge of all design and field engineering work. His duties were broadened in 1927 to include all construction activities. In 1928 his long-felt desire was satisfied when he was appointed to the position of engineer in charge of the Los Angeles Aqueduct. This position included, in addition to the maintenance and operation of the aqueduct system, the responsibility of drilling and operating more than 100 wells in Owens Valley; purchase, lease, and maintenance of more than 250,000 acres of city-owned lands and property in Owens Valley and Mono Basin; and complete hydrographic control of the water supply and runoff forecasts through snow surveys.

Notable was his work of increasing the capacity of the aqueduct from 440 cu ft per sec to 480 cu ft per sec in a period of four years. This was accomplished by a well-planned and executed program of resurfacing the bottom and sidewalls of the aqueduct to reduce friction losses.

Mr. Phillips' broad knowledge of water distribution system design, construction, and appraisal was recognized by the water works profession throughout the West. He was frequently called upon to act as consulting engineer on valuations and appraisals for private and municipal water systems. In 1928 he was engaged by the City of Fresno, Calif., to appraise a complete $2,500,000 water distribution system for the purpose of condemnation proceedings.

Mr. Phillips possessed a genial personality which won for him a wide circle of friends and made close companions of many of his associates. He was blessed with the happy faculty of seeing the sunshine in personal problems, and his employees frequently found comfort in his cheerful advice. His ability and judgment were always recognized and respected by his associates. His firm policy of honesty and justice in dealings with employees commanded the highest degree of fidelity and loyalty among his subordinates.

He was profoundly interested in younger men; his able counsel and affectionate interest left its impression on many a young man and helped him over difficult places. He was an ardent fan of the college sports and professional baseball. Hunting and fishing had only slight appeal as a recreation, although he was fond of the outdoors and spent much time studying mining prospects and large construction projects.

On October 5, 1910, he was married to Lucy Anna Batman, who, with three sons, Dr. James Emerson, Jr.; Robert Van Ness, Jun. Am. Soc. C. E., and William Franklin, survives him.

He was a member of the Blue Lodge in Indiana, belonging to the Hall of Parke Lodge No. 8, Free and Accepted Masons. He was a registered civil engineer in the State of California, and also held membership in the American Water Works Association, serving as secretary-treasurer, vice-president, president, and director of the California Section from 1935 to 1939.

Mr. Phillips was elected an Associate Member of the American Society of Civil Engineers on March 16, 1925, and a Member on October 26, 1931.

CLIFFORD MORGAN PINCKNEY, M. Am. Soc. C. E.[1]

DIED SEPTEMBER 12, 1940

The statement has frequently been made—and it probably is not an exaggeration—that in his twenty years as chief engineer of the Borough of Manhattan, New York, N. Y., Clifford M. Pinckney never refused to receive a telephone call or a visitor. Outstanding as he was in his professional capacity as civil engineer and city planner, he was never too occupied or too important to listen to one less important than himself.

There is scarcely a block in the Borough of Manhattan where Clifford M. Pinckney is not commemorated by the monuments of steel, concrete, and stone that came into being as a result of his professional knowledge, foresight, and initiative. These are important, but, to his thousands of friends in every walk of life through the width and length of the United States, Mr. Pinckney will be better remembered for his kindly, sensitive nature which he frequently tried unsuccessfully to hide under an unconcerned, carefree exterior.

The combination of his professional skill, his foresight in technical planning, and his deep-seated interest in the welfare of those who worked with him and for him make him one of the outstanding municipal executives of the present century.

Someone has remarked that a person who cares for animals cannot be wrong. Not only as president of the Humane Society of the city in which he lived, but likewise as an individual, Clifford Pinckney was passionately devoted to animals. Less than three months before his death, and while he was suffering from a serious illness, he waded through a brook in a city park to release a muskrat that had been illegally trapped.

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[1] Memoir prepared by James W. Danahy, Vice-Pres. and Managing Director, West Side Association of Commerce, Inc., New York, N. Y.

Anyone not knowing Clifford Pinckney might conceivably infer from the foregoing that he was lacking in the qualities of sternness, aggressiveness, and similar characteristics which are normally assigned to successful professional men and leaders of industry. Certain it is that he was never gruff, nor mean, nor hard. It is probable that he would have resigned his professional positions if he felt that these characteristics were essential to success; but let nobody for a moment imagine that Clifford Pinckney did not command the respect of those whose labors he directed. Providence has endowed relatively few persons with the ability to say "no" in such a way as to keep the respect and admiration of those who are refused. Clifford Pinckney was one of these few.

Those who knew him liked to think of Clifford Pinckney as a typical American, and there was considerable reason for such a characterization, inasmuch as he was a descendant of Charles C. Pinckney, one of the original signers of the Constitution. Born in New Rochelle, N. Y., on January 15, 1883, he was educated at Trinity School, Richardson School, and Woodbridge School and was graduated from Columbia University, in New York City, in the class of 1904. For a short time he worked in the Engineering Department of the New York Telephone Company, but on September 8, 1904, he entered the service of the City of New York, where he was to remain until January 31, 1938. His first job was with the Board of Rapid Transit Commissioners (later discontinued), which had charge of the East River subway tunnels being built between Manhattan and Brooklyn.

After a short stay with the Topographical Bureau of the Borough of Queens, where he did his first work on city planning, Mr. Pinckney was transferred to the Engineering Department of the Borough of the Bronx and was assigned to the construction of storm relief sewers. He was named as assistant consulting engineer of that Borough in 1910 and two years later was appointed as engineer of maintenance of highway. It was on January 1, 1914, that he was transferred to the Borough of Manhattan where, in later years, he was to become so well known. In June, 1918, he became chief engineer of the Bureau of Highways and the following January, when various departments were consolidated, he became chief engineer of the Borough. Included under his jurisdiction were sewers, highways, topography, and city planning.

Mr. Pinckney's early work in the Topographical Bureau in Queens had impressed upon him the absolute lack of city planning for the future which existed throughout greater New York. His first task, therefore, as chief engineer of the Borough of Manhattan, was to lay out a comprehensive plan for traffic relief. This plan, involving the widening of roadways, extension of main arteries, elimination of grade crossings, and removal of street-car tracks and elevated columns, was presented to the Borough President, accepted by him, and has been followed rigorously to the present time. Some idea of the magnitude of this plan may be gained from the fact that it was he who suggested the West Side Elevated Highway, the East River and Lincoln tunnels, the Park Avenue Viaduct, the Mid-Manhattan Underpass, and the East River

Drive. In addition to conceiving these ideas, Mr. Pinckney, as chief engineer, built the West Side Highway and the Park Avenue Viaduct and initiated the work of constructing the East River Drive. He likewise represented the city in the negotiations for the Manhattan approaches to the Holland Tunnel, the Lincoln Tunnel, the Queens Midtown Tunnel, and the George Washington Bridge.

Appointed by the Mayor in 1926, he was one of the members of the West Side Improvement Engineering Committee that consolidated the efforts which had been made for forty years to remove the railroad tracks from Eleventh Avenue.

Volumes would be required to obtain an adequate idea of the work he accomplished in the widening of streets, extension of various thoroughfares, rebuilding of sewers, construction of storm relief sewers and sewage treatment plants, and the removal of car tracks and elevated columns. A few of these projects, however, demand at least passing attention. Among them are the Park Avenue ramp at Thirty-Fourth Street; the overhead roadways around Grand Central Terminal; the Riverside Drive Viaduct from 155th Street to 161st Street; the removal of elevated spurs on Sixth Avenue, Thirty-Fourth Street, and Forty-Second Street; and the Canal Street, Dyckman Street, and Wards Island sewage treatment plants. With the effectuation of the principles set forth in his "Practical Plan for Traffic Relief," which had been presented to the Borough President of Manhattan in 1923, Mr. Pinckney, on January 31, 1938, resigned the office which he had held for twenty years.

With the idea of continuing his city planning activities, he became associated, on February 1, 1938, with the West Side Association of Commerce in the capacity of chief engineer and industrial consultant. He immediately initiated the laying out of a West Side Plan, the first step of which was approved by the Board of Directors of the Association and submitted to the City Planning Commission in October of the same year. His vision of city planning, as initiated in 1923, was expanded in this proposal, which called for the widening of certain downtown streets in Manhattan and for the construction of arcades over sidewalks on streets where widening was not feasible. A novel plan for the utilization of private property for the approaches to the Brooklyn-Battery Tunnel was likewise included. Subsequently, an adaptation of this same approach plan was developed by him in detail and was under consideration by the Borough President of Manhattan at the time of Mr. Pinckney's death. During the period that he was with the West Side Association of Commerce he likewise acted as consulting engineer on the paving at the World's Fair and the Randalls' Island junction of the Triborough Bridge, on the Red Hook Housing Project, and on the Fire Island Hurricane Restoration Project.

In January, 1939, Mr. Pinckney was stricken with a heart attack and from that time until his death suffered continuously from various ailments which were attributable to this attack. Only his closest associates knew, however, of the suffering which these ailments caused him. It was typical of his nature in any conversations to minimize his personal feelings.

In addition to his membership in the Society, Mr. Pinckney was a member of the National Society of Professional Engineers, New York Society of Professional Engineers, Municipal Engineers, the Society of Terminal Engineers, and the Wykagyl Country Club.

No epitaph is needed or could do justice to Clifford M. Pinckney. He did his work honestly, courageously, and with a vision to the future. He was beloved and respected by those who worked with him and for him. He left enduring monuments of stone and more enduring monuments of friendship. He made his city a better place in which to work and live. He was a real American.

On October 8, 1905, Mr. Pinckney was married to Ethel Moore of New Rochelle, N. Y., who survives him.

Mr. Pinckney was elected a Member of the American Society of Civil Engineers on July 9, 1923.

ARTHUR LESLIE PLIMPTON, M. Am. Soc. C. E.[1]

Died July 16, 1939

Arthur Leslie Plimpton was born on August 4, 1855, in the City of Roxbury (later a part of Boston), Mass., the son of Jeremiah and Sarah Davenport (Bradlee) Plimpton. He came of New England stock, and of Puritan stock. His earliest ancestor to reach the United States was John Plympton (as the name was then spelled). The exact locality in England from which he came is not known, nor is the date of his birth, but it is thought to have been about 1620. He apparently was a well-educated man, judging by his signature which is an example of good penmanship measured by modern standards. In a "Genealogy of the Family," by Levi B. Chase,[2] the opinion is expressed that when John Plympton embraced the faith of the Puritans he was ostracized by his family, who were people of quality, and this led to his embarkation to America.

Certain it is that he was a resident of Roxbury in 1641. Authentic records show that John Plympton lived as early as 1643 in Dedham, then a plantation about ten miles south of Boston. It was there that he was married in 1644. A few years later he was a factor in establishing a village that was later called the town of Medfield, Mass., a few miles south of Dedham. The house that he erected there was called the "Homestead" by succeeding generations. He was a man of consequence there, a surveyor of highways, a constable, and a fence-viewer.

In 1673 he and his family moved to what was later known as Deerfield, Mass., being attracted by the fertile valley of the Connecticut River. Almost as soon as they were well settled, war broke out with the Indians. John Plympton had joined the military organization in 1643, the first organization of its kind in the United States and still existing under the name of The Ancient and Honorable Artillery in Boston. This affiliation, together with the quality of leadership, placed him at the head of the military forces. When the war was all but ended, he was surprised in his own kitchen, taken prisoner by Indians, and eventually burned at the stake. He had been a pioneer, a man of enterprise, and unquestionably a leader. His succeeding generation quite largely settled within easy distance from the "Homestead" at Medfield.

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[1] Memoir prepared by C. Frank Allen, Hon. M. Am. Soc. C. E.

[2] Plimpton Mfg. Co. Print, Hartford, Conn., 1884.

From John Plympton, in line of direct descent, Arthur Leslie Plimpton was of the eighth generation. His father, Jeremiah Plimpton, spent his early days on his father's farm in Sharon, Mass., attending the district schools of that small town. He did some teaching during the winter sessions in Sharon and in Dedham. Realizing the insufficiency of his early education, he found opportunity to pursue his studies at the academies at Wrentham, Mass., and at Milton, Mass.

After teaching for a few years at Milton and Dorchester, Mass., he was called to the position of headmaster of the Dudley Grammar School in Roxbury, which he conducted with success. He took much pride in the fact that on an occasion when his school entered into competition with some Boston school, the award of superiority was taken by his school. On the completion of sixteen years in this position, impaired health led to his resignation. Later he entered the service of the United States Government, in a less arduous post, as an assistant assessor.

Arthur L. Plimpton was educated in the public schools of his native city, graduating from the Roxbury High School. Military drill prevailed then, as now, in Boston, and he was the Captain of the High School Company. He was awarded first prize for individual drill in a prize drill held in Boston. He then entered the Massachusetts Institute of Technology, at Boston, and was graduated with the class of 1877, receiving the degree of Bachelor of Science in Civil Engineering.

Following graduation he secured employment on the Improved Sewerage System for Boston, locally the most important project on hand at that time. Not all of his work was routine, for he was put in charge of several important undertakings that required able handling. These involved a study of the sub-strata of Moon Island Bar, and the planning of a system of triangulation to secure data essential for tunnel location dependent partly on length, which resulted in the adoption of a tunnel more than a mile and a quarter long.

In 1886, Mr. Plimpton entered the employ of the Metropolitan Street Railway Company, a corporation of some thirty years standing, for which he did all the engineering work. Two years later he was given the title of principal assistant engineer for the West End Street Railway, a fairly ambitious company organized in 1887. For this company, he laid out the original lines for 273 miles of track. He later was given the title of civil engineer, the term chief engineer not being used at that time.

Disregarding technical distinctions as to priority of use, the West End Street Railway was a pioneer in the construction and operation of electric railways. In the days of horsecars, the track consisted of thin rails laid on stringers, with short-grooved rails of cast iron on the inside of curves, and flat rails on the outside on which the wheel flanges rode or slid. The necessity for a better rail became emphasized with the introduction of the electric railway. The girder type of rail had already made its appearance with varying forms of supports, with chairs resting on ties or masonry piers, and with several shapes of head and base, the aim being in part to facilitate traffic with the steel tires then in use on vehicles. The outcome has been the deep 9-in. ($8\frac{25}{32}$) girder rail supported on ties with space for paving blocks either inside or outside the rail.

Mr. Plimpton's experience during this formative period gave him his opportunites and he became an engineer of superior judgment and capability on street railway track; he was regarded highly both by his company and by his contemporaries. In 1897, the West End Street Railway Company was leased to the Boston Elevated Railway Company and he continued with this company as the civil engineer-in-chief of the surface lines. In 1918, after more than thirty years of service, he was appointed advisory engineer and in 1934 was granted age retirement with pension.

For a number of these years, Mr. Plimpton also successfully operated a cranberry meadow in a neighboring town. During the World War, the labor situation made it difficult to continue its operation to advantage, and it came into disuse and finally was sold.

Mr. Plimpton was a resident of West Roxbury, Mass., for more than forty years, and although he was not especially active in local affairs, he had been a member of the Highland Club and the Citizens' Association, as well as of the First Parish West Roxbury, the Unitarian Club, and the Unitarian Laymen's League. He was a good citizen. In a larger field he had been a member of the Boston Society of Civil Engineers, the American Electrical Railway Engineering Association, and the National Geographic Society.

He was quiet and unostentatious in temperament. He had the faculty, unusual at least in degree, of feeling and expressing happiness at the good fortune of his friends in travel or otherwise, and sharing their pleasures with them. He was a good neighbor.

Arthur Leslie Plimpton was married on December 30, 1890, to Lucy Ella Richardson, who died on February 26, 1930. He is survived by a son, Herbert R. Plimpton, a daughter, Gertrude R. (Mrs. William E. Crocker), three grandsons, and two granddaughters.

Mr. Plimpton was elected a Member of the American Society of Civil Engineers on May 6, 1896.

VICTOR HENRY POSS, M. Am. Soc. C. E.[1]

DIED NOVEMBER 16, 1940

Victor Henry Poss, the son of Henry Poss and Pauline (Johanni) Poss, was born in St. Louis, Mo., on July 27, 1870. As a boy Mr. Poss attended the St. Louis Manual Training School, graduating in 1887. After a year of travel and study in Europe he entered Cornell University at Ithaca, N. Y., graduating in 1892 with the degree of Civil Engineer.

On March 1, 1898, in St. Louis, Mr. Poss married Mabel Alice Lawson. Mrs. Poss and an only son Lawson Victor Poss survive him.

For nine years after graduation from Cornell, Mr. Poss was engaged as designing engineer with the U. S. Engineer's Office at St. Louis, in connection with floating equipment on the Mississippi.

From 1901 to 1903 he was engaged as assistant engineer with the Wisconsin Bridge Company at Milwaukee, Wis. In 1903 he came to San Francisco, Calif., as chief engineer for Reid Brothers, Architects. In this capacity, both before and after the fire and earthquake of 1906, he had charge of the design of many of the important structures in San Francisco. In 1909 and 1910 he traveled in the Orient and Europe for pleasure and education. From September 1910 on, he was engaged in private practice in San Francisco, as Consulting Engineer.

During World War 1 he designed the hull of the concrete ship "Faith," and from 1918 to 1921 acted as chief engineer and general manager of the San Francisco Shipbuilding Company.

Mr. Poss was a member of the Delta Phi Fraternity, the San Francisco Section of the Society, the Cornell Association of Civil Engineers, the Structural Engineers Association of Northern California and the Engineers Club of San Francisco.

His outstanding characteristic was thoroughness and without exception a conscientious execution of all work he undertook, to the verge of the meticulous. His intimates appreciated his universal kindness and reticence. It is regrettable that intimate friendship with one so retiring is essentially limited to so few, but to these few the loss of such an one is very real indeed.

Mr. Poss was elected a Member of the American Society of Civil Engineers on April 5, 1910.

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[1] Memoir prepared by Frederic F. Hall and John B. Leonard, Members, Am. Soc. C. E.

ALEXANDER POTTER, M. Am. Soc. C. E.[1]

DIED JUNE 20, 1940

With the passing of Alexander Potter, there is lost an engineer, distinguished among engineers for his forthrightness, aggressiveness, pioneer spirit, and individualism.

Born at Gibralter on January 18, 1866, the son of a British soldier in the Seventy Eighth Highlanders, James Swanston Potter, and Elizabeth (Stewart) Potter, he was brought to Halifax, Nova Scotia, at the age of four. Upon graduation from the local high school, Mr. Potter was indentured in good British fashion to the city engineer for three years' service as an engineering student. In 1886, before entering Lehigh University, at Bethlehem, Pa., Mr. Potter was employed by the Canadian Pacific Railway Company for eight months; then he worked on the Santa Fe Railway for one year, and with the city engineer of Kansas City, Mo. His practical background served him in good stead when he entered Lehigh University in September, 1888, for he completed the undergraduate curriculum in three years and received the degree of Civil Engineer in 1890. His professor, the late Mansfield Merriman,[2] M. Am. Soc. C. E., resented this speeding up of scholasticism at the time, but in later years relented his former attitude.

In fifty years of professional practice in municipal engineering, following his graduation, Mr. Potter served more than one hundred and fifty municipalities in various capacities, primarily as an expert in sanitation and water supply. Not a few of these municipalities engaged his services repeatedly over many years. Early in his career he was called upon by a group of municipalities in Union and Essex counties, New Jersey, for advice in connection with the Joint Trunk Sewer. Characteristic of his indefatigable spirit and initiative, when he found that lack of legislation prevented the formation of a sewer district in New Jersey, he worked actively with lawyers in framing the necessary laws and personally lobbyed for them in Trenton. For forty-two years he was continuously associated with this group of municipalities composing what has been termed "The Joint Meeting." An enlarged group, including the original Joint Meeting, in the later 1920's, constructed the supplemental Joint Trunk Sewer in Union and Essex counties. The original joint sewer project was the first in the State of New Jersey and the forerunner of many similar joint enterprises. The form of contract and method of payment in which Mr. Potter's counsel was employed to a large degree have been the basis of many other similar undertakings.

The cosmopolitan professional experience was presaged by cosmopolitan training. Mr. Potter was employed in Mexico, Australia, and Cuba. At the close of the Spanish-American War, he was commissioned by the Secretary of War of the United States to make a survey of the sanitary conditions at

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[1] Memoir prepared by Seth G. Hess and Clinton L. Bogert, Members, Am. Soc. C. E.

[2] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1658.

Santiago de Cuba. In 1912, he was appointed by the Cuban Government as one of a commission of three to investigate a $15,000,000 sewer and paving contract in Habana. In 1914, by act of the Cuban Congress, he was designated to prepare plans and supervise the construction of the water works project at Santiago de Cuba. For nearly twenty years, Mr. Potter continued to serve the Cuban Government on sanitary, water supply, and hydraulic problems in the principal cities of the island.

During the first World War, he served the United States Government as sanitation engineer of the large explosive plant at Nitro, W. Va., and consultant engineer for Camp Custer, at Battle Creek, Mich., and Camp Upton, at Yaphank, N. Y.

Notable as an expert in litigation, he was a most enthusiastic witness, and gave testimony in clearest detail, building up a concise, convincing story. His lengthy and successful appearance on behalf of the contractor on the New Orleans (La.) sewers when the late Rudolph Hering,[3] M. Am. Soc. C. E., appeared on the side of the city is outstanding in sewer litigations.

In 1936, Lehigh University took note of his long and outstanding engineering career and loyal service to the university, as alumnus and trustee, by conferring upon Mr. Potter the degree of Doctor of Engineering. He was also honored by designation by Herbert H. Lehman, Governor of the State of New York, as unsalaried commissioner on the Rockland-Westchester-Hudson River Crossing Authority, of which he became vice-chairman.

Mr. Potter's membership in technical societies included, among others, American Association of Engineers, American Association for the Advancement of Science, American Association for Municipal Improvement, American Water Works Association, American Public Health Association, Engineering Institute of Canada, Engineering Society of Pennsylvania, National Municipal League, New England Water Works Association, Ohio Engineering Society, Oklahoma Society of Civil Engineers, Association of Managers of Sewage Disposal Works (Great Britain, Honorary Member), and New Jersey Sewage Works Association. His clubs included New York Rotary (Director), Elks, Lehigh University, Canadian, Railroad-Machinery, and El Country de la Habana.

No memoir of Mr. Potter would be complete without reference to one who worked devotedly with him and whose judgment he valued so highly. Florence Dangerfield Potter, his wife, was intimately associated with all of his business affairs. As a Bachelor of Laws and a member of the New York Bar, her advice was invaluable to him. She died in 1932.

Mr. Potter possessed a vigorous and colorful personality. His keen sense of humor, coupled with a seemingly inexhaustible fund of tales of his experiences, earned for him the reputation of a pleasant companion and interesting associate. He was an independent thinker, courageous and fearless. Habit, usage, or custom were brushed aside with impunity when he was convinced he was right. He willingly took issue with any authority, in his specialty, and courageously opposed any agency or officer no matter how powerful.

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[3] For memoir, see *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 1350.

His attitude toward contractors, operating under his direction, was characteristic of his fearlessness and vigor. He worked on the premise that a contractor doing honest work was entitled to an honest profit. His specifications were written to relieve the contractor in so far as possible of the hazards of gambling and doubt. He was insistent that a contractor should be paid not merely for quantity but for quality of work performed. When underground conditions could not be foreseen accurately, he was adamant with the governing bodies, insisting that the contractor should be compensated for the more difficult work which could not have been anticipated. Without compromising his insistence upon good work well done, he would nevertheless fight "tooth and nail" to obtain compensation for a contractor who encountered unforeseen difficulties resulting in expenditures that could not have been anticipated.

His keen mind, aggressive spirit, good engineering background, awareness of every advance in his engineering specialties, and assimilation of legal knowledge made him a most competent engineer and a formidable expert witness.

He is survived by two sisters and three brothers.

Mr. Potter was elected a Junior of the American Society of Civil Engineers on January 4, 1888; an Associate Member on January 6, 1892; and a Member on April 13, 1931.

JOSEPH HENRY PRIOR, M. Am. Soc. C. E.[1]

Died July 1, 1940

Joseph Henry Prior was born in Chicago, Ill., on May 15, 1877, the son of Patrick and Katherine (Allen) Prior. His father and mother were both natives of Ireland. He was educated in the public grade and high schools of Chicago, and studied advanced mathematics in the University of Chicago. He also attended courses in Armour Institute of Technology, in Chicago.

Mr. Prior had a long and notable career in Chicago and Illinois. In 1895 he was employed by the Illinois Steel Company, and later by the late George S. Morison,[2] Past-President, Am. Soc. C. E., consulting engineer, as a rodman and draftsman. In October, 1897, he entered the service of the Chicago, Milwaukee and St. Paul Railway Company under the late Onward Bates,[3] Past-President and Hon. M. Am. Soc. C. E., as a shop inspector and draftsman. In June, 1902, he was employed by the late Ralph Modjeski, M. Am. Soc. C. E., consulting engineer, as shop inspector and draftsman on the building of the Thebes Bridge over the Mississippi River.

Mr. Prior returned to the Chicago, Milwaukee and St. Paul Railway Company in November, 1904, as a bridge designer under the late Charles F. Loweth,[4] Past-President, Am. Soc. C. E., engineer and superintendent of bridges and buildings. In August, 1905, he was promoted to assistant engineer, and in June, 1912, was made engineer of design in charge of a large force. Some very important structures were designed and detailed under his direction, among which were the bridge over the Des Moines River in Iowa; the Hastings and Dakota Division track depression work in Minneapolis, Minn.; the Irving-Howard track elevation in Chicago; the Spokane (Wash.) grade separation; the bridge over the Missouri River at Mobridge, S. Dak.; several steel viaducts on the Puget Sound Line; the drawbridge over the Kinnickinnic River in Milwaukee, Wis.; and the drawbridge over the North Branch of the Chicago River at Kinzie Street, Chicago.

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[1] Memoir prepared by T. L. Condron, I. F. Stern, Members, Am. Soc. C. E., and W. S. Lacher, Assoc. M. Am. Soc. C. E.

[2] For memoir, see *Transactions*, Am. Soc. C. E., Vol. LIV, June, 1905, p. 513.

[3] For memoir, *loc. cit.*, Vol. 102 (1937), p. 1484.

[4] For memoir, *loc. cit.*, Vol. 101 (1936), p. 1488.

In April, 1914, Mr. Prior resigned from the Bridge and Building Department of the Chicago, Milwaukee and St. Paul Railway Company to become assistant chief engineer of the Public Utilities Commission of the State of Illinois (later the Illinois Commerce Commission), being advanced to chief engineer in 1916. During this time with the Public Utilities Commission, he did much pioneer work on the subject of Valuation of Public Utilities, and later was called upon as a consulting engineer and arbitrator to decide numerous contentions in connection with valuations made for the determination of rates.

From 1916 to 1925 he engaged in private practice as a consulting engineer with headquarters in Chicago, being retained as an engineering expert by various clients, including the Chicago City Council Committee, the Chicago Board of Local Improvements, and the Chicago Plan Commission, in connection with important municipal improvements. In 1925 Mr. Prior accepted the appointment as chief architectural engineer of the Chicago Board of Education, which position he continued to hold until his death.

In May, 1923, he was appointed a member of Committee of Examiners for the Registration of Structural Engineers, State of Illinois, on which committee he served until his death, filling the office of chairman during the last three years.

Mr. Prior was a member of the American Railway Engineering Association, the Western Society of Engineers, and the Library Board of the Lewis School of Aeronautics at Lockport, Ill., to which school his very extensive library was given.

He was esteemed as an engineer of exceptional ability and integrity, but did not confine himself to engineering alone. He was a student in the best meaning of the word during his entire life and was particularly well informed with regard to economics, as well as rules of legal procedure, especially those applying to public bodies, on which matters he was frequently consulted. He was the author of several articles, in engineering publications, which were descriptive of structures designed under his direction.

Mr. Prior was an active communicant of St. Ambrose Roman Catholic Church in Chicago. He never married and is survived by a sister, Mrs. Charles T. Byrne.

Mr. Prior was elected a Member of the American Society of Civil Engineers on September 2, 1914.

RALPH FENNO PROCTOR, M. Am. Soc. C. E.[1]

Died May 29, 1940

Ralph Fenno Proctor—one easily writes the name; but how difficult it is to put into proper phrase all that the name connotes. First, there is the man himself, the instincts and qualities that differentiated him from other men; then there is his work, in and by which these instincts and qualities found expression in a product and a permanence not granted to the man himself.

Engineering is a fascinating field of human enterprise. It leads one (or may, if he follows the path of opening opportunities) into so many fields, so different though so closely related, so alluring that the excitement never ceases, and the anticipation of new achievements ever beckons on and on, away from the accomplished and the realized of the present into the unknown future of the better days that are yet to be. It is an incurable fever in the brain: A tireless taskmaster to the will, but a priceless possession at the end of the weariest day; a soul-satisfying joy at the least, because of definite advance and accomplishment.

How does all this concern Colonel Proctor? Because he heard this clear call of engineering in his earliest youth, and obedient to its compelling urge followed it into many fields, each quite distinct, but each allied to, and subject to, this spirit of engineering. In each of them he strove to raise yet higher the standards, based, as all engineering is (or should be), upon the application of definite, positive, and unchanging laws to science, industry, or business—laws as yet not fully formulated or understood, but ever moving toward their final and fixed acceptance by all.

Ralph Fenno Proctor was of New England ancestry. Born in Dorchester, Mass., on March 27, 1878, the son of Frederick E. and Sarah (Fenno) Proctor, and cultured in that New England atmosphere which has contributed so materially to the progress and prosperity of the country, he developed a strong physique on his grandfather's farm at Athol, Mass., and laid the foundation of a broad and exact knowledge in the schools at Newton, Mass. This early preparation was completed at Cornell University, in Ithaca, N. Y., from which institution he was graduated in 1901.

Immediately after graduation he joined the Great Lakes survey as chief of party, continuing as such for two years, and was engaged in surveys, soundings, studies, and investigations incident to the wider usefulness of these bodies of water.

As so often happens, Mr. Proctor made very deep and lasting friendships at the university, and in 1903 these friendships were more fully cemented in the firm of Williams and Whitman, and later of Whitman, Proctor and Potts, Inc., of New York, N. Y. This organization was actively engaged as consulting and constructing engineers in the design and building of sanitary systems and public water supplies, in which it specialized. Among the municipal works undertaken and completed during this association were sanitary works for Havre de Grace, Md.; Newton and Morristown, N. J.; Waverly and Ithaca, N. Y.; and Ravenna and Leetonia, Ohio, as well as for other communities. A project for the U. S. Government and a water supply for the President, Theodore Roosevelt, for his home at Oyster Bay, Long Island, N. Y., were completed successfully by this firm.

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[1] Memoir prepared by Herbert M. Knight, M. Am. Soc. C. E.

This association might, and probably would, have continued, but for the drafting of some of its members to engage directly in the design and construction of certain very large municipal sewage disposal systems. Although this association had continued for only six years, Mr. Proctor had gained through it a first-hand knowledge of the elements of design most essential in connection with the conduct and satisfactory completion of public works, a preparation of prime importance to the wider field he was later to cover in his combined constructional and financial activity.

This foundation, as to the relationship of sound design to adequate financial return, was further amplified and established when Mr. Proctor became, in 1909, chief engineer and general manager of the contracting and engineering firm of A. L. Register, of Philadelphia, Pa. In this association Mr. Proctor's range of activity was much further enlarged, not only in completing large construction works but in the problems of raising funds and financing such projects.

In 1913, Mr. Proctor was appointed chief engineer of the Maryland Casualty Company, of Baltimore, Md. He was a pioneer in a new field of engineering. Few surety companies, despite their close association and involvement in contracts, had engineers on their staffs, and most companies had scant respect for engineers in this capacity; but Mr. Proctor brought to this, as yet unoccupied, field of engineering a knowledge of contracting and a knowledge of finance that admirably fitted him for this responsible and untried field.

Always Mr. Proctor combined three functions, three personalities—an engineer, a contractor, and a financier. In handling many of the problems he was called upon to solve daily, he used these three distinctive elements for the safety and security of his company, for the safety and indemnity of the public body being protected, and for the assistance and profit of the contractor being guaranteed. To have successfully administered his position for so many years, in view of the often antagonistic attitudes of the chief parties to constructional contracts, shows the tact, as well as the ability, of Mr. Proctor.

The World War of 1914–1918 marked another phase in his professional life. In May, 1917, shortly after this country entered the war, Mr. Proctor was released by the Maryland Casualty Company, and was commissioned a Major in the Quartermaster Corps and appointed constructing quartermaster in charge of the construction of Camp Meade, Maryland. Upon the completion of this project, he was given charge of the construction of the Curtis Bay Ordnance Depot, also in Maryland, and was promoted to the grade of Lieutenant Colonel. At the termination of the war, Colonel Proctor resumed his position as chief engineer and assistant to the president of the Maryland

Casualty Company and shortly thereafter was made a vice-president in charge of the bonding department, continuing in that capacity until 1934. A few years after the war Colonel Proctor had charge of the construction of the handsome group of buildings which serve as the home of this company.

In 1934 he resigned his position to assume that of manager of the fidelity and contract bonding department for the Association of Casualty and Surety Executives. This is a nation-wide organization engaged in the wider operation and development of those types of protection known as casualty and surety bonds, of which the latter contract bonds form an important part. To a very considerable degree this new association was a well-earned promotion for Colonel Proctor. His activities with the Maryland Casualty Company had covered largely that company's own operations, while in his new association his activities were national. They involved policies and procedures affecting all surety companies. This entailed a great amount of traveling and an enlargement of the Colonel's already wide acquaintance with the engineers and contractors active in the important developments of the time. In this new position he was finding an ultimate of accomplishment and satisfaction.

Among the outstanding characteristics of Colonel Proctor was an abounding good-fellowship—a friendliness, and a freedom of approach to men and matters that were both unusual and helpful. With Colonel Proctor this readiness of approach and response to understanding were innate and spontaneous. These qualities grew out of the man and his wide and intimate association with engineering and construction under both favorable and unfavorable aspects, and both in his own experience and in that of the contractors with whom, in their darker and less successful moments, he had dealt. This wide sympathy and understanding with, and of, the problems, the difficulties, and the possibilities inherent in the completion of large construction enterprises constituted one of Colonel Proctor's most valuable assets, and was of incalculable value to the large financial interests with which he was associated, and also to the many contractors to whom his advice and helpful counsel were of inestimable assistance.

It is a difficult, not to say impossible, task to appraise adequately the service rendered by any man, particularly an engineer, to his own or to a succeeding generation. To some engineers is given the task, and the honor, of conceiving and completing monumental and epochal structures—a mighty bridge, spanning a seemingly impassable chasm; an extensive system of municipal improvement, safeguarding and increasing the health and the happiness of many people crowded into narrow and confining areas; a large tunnel, boring deep beneath wide rivers, making separated communities one; a far-flung system of communication making of scattered states a great and a unified country; or a colossal dam, holding back the torrents from the hills and bidding them carry light and heat and motion to the millions in the valleys. These, and many other phases of engineering and construction, have brought to the forefront men of exceptional ability and fitness, whose names will be emblazoned forever high in the annals of progress and achievement. They constitute the alluring, the commanding, the spectacular. Of them, and what they have rendered, the engineering profession may well be proud.

Then there are other elements of engineering, less spectacular, but nonetheless important, and among these elements is that of the close relationship between construction and finance. Money and engineering are inseparably united. They can never be permanently divorced. Too often, engineers, under the alluring spell of some great and attractive engineering program, have forgotten that there is no warrant for any enterprise that cannot justify itself. Honesty to fundamental physical laws is the bedrock of technical engineering; honesty to fundamental financial laws should likewise be the bedrock of constructional activity.

Faithful to these two foundations of all true engineering and of all true enterprise, Colonel Proctor worked and strove. His was the task of clearly and carefully analyzing the many factors incident to varied public contracts; to determine the correctness and completeness of the assumptions made by the many contractors seeking his guarantee; to determine the safety and reliability of their standing; to outline and determine the policies and practices of his company, and of many companies, in the conduct of contract underwriting and bonding; and, in that wide, difficult, and little known field, to establish such principles and procedures as should eventually lead to a more scientific and standard course and to a better and more friendly relationship as between the many, and often antagonistic, interests involved. In this work and effort Colonel Proctor was not only a pioneer but a positive contributor, and the result of his thought and labor will be permanent. As the intimacy and interdependence of engineering and finance become recognized more and more, so the contribution of Colonel Proctor will be increasingly appreciated.

On November 16, 1906, he was married to Ilka Slingluff who survives him. Also surviving is a son, Ralph F., Jr., and a grandson, Ralph F., the Third.

He was a member of the Episcopal Church, and Tau Beta Pi fraternity.

Colonel Proctor was elected a Junior of the American Society of Civil Engineers on June 3, 1902; an Associate Member on October 4, 1905; and a Member on March 2, 1915.

CHARLES DE LA CHEROIS PURDON, M. Am. Soc. C. E.[1]

DIED JUNE 3, 1940

Charles de la Cherois Purdon was born in Belfast, Northern Ireland, on October 6, 1850, the son of C. D. Purdon and Jane M. (Calvert) Purdon. He was descended from James Purdon, who was settled in Ireland during the reign of Queen Elizabeth. His middle name, de la Cherois, was that of his paternal grandmother. She was a descendant of a French Huguenot family, who were settled in Picardy, France; at the Revocation of the Edict of Nantes they fled to Holland. In 1698, King William III, of England, induced this family with others to go to Ireland to establish the cultivation of flax and the manufacture of linen.

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[1] Memoir prepared by F. G. Jonah and F. W. Green, Members, Am. Soc. C. E.

Charles Purdon was educated in the private schools of Belfast and completed the mathematical course of Queens University, in that city, under a private tutor. His first employment was in the office of an uncle, who was an architect. There he acquired proficiency as a draftsman and some knowledge of architecture.

At an early age he came to New York, N. Y., to enter business with some relatives who were in the linen business, and it was thought that he might make a salesman for their house. He records in his *Memoirs* a trip that he made to Baltimore, Md., and New Orleans, La., during the Reconstruction Period, and was much impressed by the conditions as he saw them in the South at that time. He decided, however, that he would never make a salesman, so resigned and went to Nova Scotia, Canada, where he obtained employment on the construction of the Intercolonial Railway, later a part of the Canadian National System. His headquarters were at Truro, Nova Scotia, and from 1870 to 1872 he was an axman, a walking boss for a contractor, and assistant engineer on 8 miles of heavy construction work.

From 1872 to 1874 he was an assistant engineer in the Public Works Department of Canada and had charge of surveys on the St. Lawrence River for the enlargement of canals. During the winters this survey was made on the ice, with temperatures as low as 40° below zero.

At the conclusion of that work he made a visit to Ireland, but returned to Canada in 1875. Finding no employment in Canada, he went to Texas with a friend and they jointly bought some land for a farm. For about five years they devoted themselves to the task of making a living out of the ground.

In 1880 Mr. Purdon secured employment as a draftsman, transitman, assistant engineer on construction, and a locating engineer on 237 miles of line for the Texas and St. Louis Railway Company, commonly called the "Cotton Belt," and later the St. Louis Southwestern Railway Company. It is interesting to recall that it was with this railway, as chief engineer and consulting engineer, that he closed his professional career. It will be noted that this period was that of the greatest activity in railroad construction in the Southwest, following the stagnation incident to the Civil War and Reconstruction Period, and it was about 1880 when railroad development began on a very active scale in the Southern and Southwestern states. Mr. Purdon's service was continuous for the 50-yr period from 1880 to his retirement in 1930 on this work of railroad building and maintenance in the West and Southwest.

In 1885, 1886, and 1887 he was with the St. Louis-San Francisco Railway Company as an assistant engineer on location and construction of the line between Monett, Mo., and Paris, Tex. He designed and had charge of the construction of the important bridge over the Arkansas River at Van Buren, Ark., which he described fully in a paper[2] published by the Society. In 1886–1887 he was in charge of the construction of the bridge, which he had designed, over the Red River near Arthur City, Tex., about 16 miles north of the important town of Paris. On completion of that work Mr. Purdon was in the employ of the Cotton Belt Railway for a year and a half, building a number of branches in Arkansas, Texas, and Louisiana.

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[2] *Transactions*, Am. Soc. C. E., Vol. XX, January–June, 1889, p. 151.

From 1888 to 1890 he was resident and division engineer on the Cumberland Valley Railroad, a branch of the Louisville and Nashville Railroad, and made surveys for other extensions in Tennessee and Alabama. Upon completion of this work, he held the following positions with the Atchison, Topeka and Santa Fe Railway Company: 1890 to 1893, bridge engineer; 1893 to 1895, resident engineer of the Chicago Division; 1895 to 1897, principal assistant engineer in charge of the improvement of bridges and trestles; and 1897 to 1901, assistant chief engineer in charge of general improvements.

His work as a bridge engineer on the Santa Fe Railway was outstanding. He was engaged for that duty by James Dunn, the chief engineer of the Santa Fe. He made a complete examination of the metal bridges on the Santa Fe System and prepared plans and specifications for stronger and better bridges. They began building new bridges after his specifications in 1893, and in the six years following a total of 844 spans of bridging were built, aggregating 43,000 ft in length, or slightly more than 8 miles. As late as May, 1933, all of the bridges then built were still in service and carrying the Santa Fe's heaviest power at high speeds. He introduced steel water tanks and ballast deck bridges while on the Santa Fe and was active in grade-separation projects on the Santa Fe in the City of Chicago, Ill.

From 1901 to 1909 Mr. Purdon was connected with the St. Louis-San Francisco Railway Company in various capacities, becoming engineer of maintenance of way, chief engineer, and consulting engineer, during which time many bridges and important structures were built from his plans and under his immediate direction. While with this company he performed a notable engineering feat in connection with the bridge of the New Orleans, Texas and Mexico Railway, a subsidiary of the St. Louis-San Francisco Railway, at the Atchafalaya River in Louisiana. One of the shore piers of that very important structure had been pushed forward by a caving bank, so that there was no bearing for the end of the span, which was temporarily carried on falsework. Mr. Purdon sank two cylinder piers beyond the limits of the pier which had moved, connecting them with a very heavy, deep girder, and rested the end of the span on that girder.

For a time in 1909 and 1910 he was in the employ of the Missouri Pacific Railroad Company on special work, and in 1910 became chief engineer of the St. Louis Southwestern Railway (Cotton Belt), with which company he continued as chief engineer, chairman of the Valuation Committee, and consulting engineer until his retirement on January 1, 1930.

Mr. Purdon had a very extensive acquaintance among the engineers and contractors of the Middle West, and had throughout these years many young engineers in his employ for whose welfare and advancement he was always particularly solicitous. He had been a member of the Engineers' Club of St. Louis since 1901 and was an honorary member of the St. Louis Engineers' Club after 1930. He served as President of the St. Louis Section of the Society and also of the St. Louis Engineers' Club. He was a member of the American Railway Engineering Association and served with distinction on its Committee for Iron and Steel Structures for a period of twelve years.

Mr. Purdon had a fine sense of proportion and a keen Irish wit, which enabled him to make the best of any situation, gave him a keen enjoyment of life, and doubtless contributed in no small measure to the lengthy span of years which he attained. He was an Episcopalian, a Mason, and a Knight Templar.

He was married on May 11, 1887, to Jennie Arthur, who, with a daughter, Eleanor, and a son, Arthur, survives him.

Mr. Purdon was elected a Member of the American Society of Civil Engineers on March 3, 1886.

JOHN WHEELER REID, M. Am. Soc. C. E.[1]

DIED MAY 26, 1940

John Wheeler Reid was born on November 1, 1872, at Grand Rapids, Mich., the son of Andrew F. Reid and Nettie (Beardsley) Reid. The family moved to Detroit, Mich., when John was six months old, and he remained there throughout his entire life. He attended the Clay, Irving, and Jefferson grade schools, attained his high school diploma by attending night classes at the old Central High School, and in 1897 was granted the degree of Bachelor of Laws from the Detroit College of Law.

In 1900 Mr. Reid was appointed as draftsman in the city engineer's office of Detroit, where successive promotions advanced him to assistant engineer. In 1917, as a result of his excellent work in drafting grade separation agreements, he was again promoted—this time to engineer of grade separations and bridges. In 1922, the resignation of Clarence W. Hubbell, M. Am. Soc. C. E., as city engineer, caused a vacancy to which Mr. Reid was promoted. He had filled this position but one year when he was appointed commissioner of public works. He served in this capacity for seven years, until his resignation in 1930. In 1932 he was elected state senator and served two terms. As senator, his service was marked by the same conscientiousness that was so evident in everything that he undertook.

The chief characteristic of Mr. Reid was his serious, methodical, and persistent forcing of the task in hand; and, as has been noted, the task was always one of public service. He was a small man, little gifted as a conversationalist, but of rock-bound integrity and, with experience, developed an exceptional capacity for hard work. He particularly demonstrated his ability to carry forward huge programs of municipal developments in his position of commissioner of public works. During his regime, literally hundreds of miles of sewers, pavements, and sidewalks were constructed, and the character of the work was improved and the costs were rendered more satisfactory to the city. During this period, the street and alley cleaning and collections were subject to improved control, while the great program for sewage treatment, completed later, was considerably advanced. Also, the municipal airport was conceived and completed at this time.

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[1] Memoir prepared by Martin R. Fisher, M. Am. Soc. C. E., and L. C. Wilcoxen, Assoc. M. Am. Soc. C. E.

On June 6, 1900, Mr. Reid was married to Maude Evelyn West who survives him. He is also survived by two sisters, Mrs. Henry G. Wallace and Mrs. John J. Hafner, and a brother, Bennett.

Mr. Reid gave considerable attention to outside interests. He was active in the affairs of the Society and the American Society for Municipal Improvements, serving as National president of the latter in 1929. He was a member of the Michigan Engineering Society and was an Elder in the Redeemer Presbyterian Church. Politically, he was a staunch Republican.

Mr. Reid was elected a Member of the American Society of Civil Engineers on September 10, 1923.

WILLIAM EDWARD ROLFE, M. Am. Soc. C. E.[1]

Died July 15, 1940

In reviewing the life story of a man whose entire professional career was spent in the service of his community, some striking feature of that life, or some particular phase of character, will be found that has distinguished him in the minds of his friends and associates, and endeared him to all of them.

Such a man was William Edward Rolfe, of whom his friends and associates are unanimous in saying that his was a pleasing personality, an alert mind with a keen sense of humor, and that he was a man of unswerving loyalty and integrity.

He was born in St. Louis, Mo., on January 1, 1874, the son of John Osgood and Caroline (Lecraw) Rolfe. He was graduated from Washington University in that city in 1895 with the degree of Bachelor of Science in Civil Engineering. That same year he was employed as an assistant engineer of distribution in the Water Department of St. Louis, and worked in that section of the St. Louis Water Department until 1911, when he was appointed assistant water commissioner. During those sixteen years the mileage of pipe in the distribution system increased from 462 to 916; and numerous changes were made in the design and manufacture of cast-iron pipe, and special castings in fire hydrants, stop and check valves, and meters—all of which came under his direct supervision.

His work was necessarily an essential part of the activities of the department during the years following 1903, when the purification of the water supply was in progress, especially in designing the arrangement of all piping necessary for the application of chemicals to the raw water.

He was made assistant water commissioner in 1911 and was in direct charge of all construction work in the department. Also, he exercised general supervision over the operation of the various sections and divisions of the department.

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[1] Memoir prepared by Edward E. Wall, Hon. M. Am. Soc. C. E.

In 1913 he accepted the position of associate to the president of the Board of Public Improvements of St. Louis, which later became the Board of Public Service, the scope of whose authority, duty, and responsibility was greatly extended and enlarged. Mr. Rolfe fulfilled the duties of this position to the satisfaction of the public and with credit to himself.

Edmund R. Kinsey, M. Am. Soc. C. E., President of the Board during this time, made the following statement:

> "For twenty years William E. Rolfe held the position of Associate to the President; his duties, his ability and his personality made him an associate in fact as well as in name.
>
> "His task was largely administrative in a department which, during his period of service, designed and executed nearly $100,000,000 worth of engineering projects.
>
> "He had in high degree a faculty not often possessed by engineers, the ability to write clear, concise English and of making his language definitely convey the meaning intended.
>
> "His was a good example of the richer life of an engineer, who adds cultural attainments to technical accomplishments."

After leaving the service of the City of St. Louis, Mr. Rolfe conducted a survey of engineering salaries in 1933 for the Joint Council of the Associated Engineering Societies of St. Louis. In 1934 he was engineer for the St. Louis Regional Planning Commission and was office supervisor for this Commission for three years—until November, 1938. On January 1, 1939, he joined L. Roy Bowen and Associates, an organization of civil engineers and architects, working with them until his death.

Mr. Rolfe was an active member of the Engineers' Club of St. Louis for more than thirty years, and served as President in 1920. From 1918 to 1933 he was editor of the *Journal* of the Engineers' Club of St. Louis, a monthly publication containing news of engineers, current events of engineering interest, papers written by engineers, and reviews of engineering books and articles.

He was President of the Washington University Alumni Association in 1915, and President of the Washington University Association in 1917 and 1918.

He was a charter member of the Missouri Beta Chapter of the Sigma Alpha Epsilon fraternity organized in 1892, and was awarded membership in Sigma Xi, an honorary scientific fraternity, in 1920.

He represented Associated Engineering Societies of St. Louis on American Engineering Council of the Federated American Engineering Societies from 1920 to 1923, and was one of the four vice-presidents from 1920 to 1922.

Mr. Rolfe was a member of the Compton Hill Congregational Church in St. Louis, and was superintendent of the Sunday School in 1910 and several years thereafter. He organized a class of girls in 1915 and was their teacher for more than ten years. This class still meets frequently and in the spring of 1940 held a reunion at Mr. Rolfe's home to celebrate the twenty-fifth anniversary of its organization.

Throughout his life Mr. Rolfe was a lover of the beautiful in literature, art, and architecture. His particular hobby was photography, which he studied and practiced assiduously long before the introduction of the modern kodak.

His sense of humor and thorough knowledge of human nature proved to be of great value to him in his official life, helping him materially to administer justice in dealing with mooted questions, such as interpreting specifications and settling disputes between inspectors and contractors.

He guarded zealously the trust that had been placed in him, and no influence or possibility of personal gain ever caused him to waver from the soundest engineering practice.

He was married in 1899 to Mary Alice Bradley, who, with one son, William Douglas Rolfe, survives him.

Mr. Rolfe was elected a Member of the American Society of Civil Engineers on January 6, 1915.

THOMAS FITCH ROWLAND, JR., M. Am. Soc. C. E.[1]

DIED JANUARY 12, 1939

Thomas Fitch Rowland, Jr., the son of the late Thomas Fitch Rowland,[2] Hon. M. Am. Soc. C. E., and Mary Eliza (Bradley) Rowland, was born in New Haven, Conn., on July 30, 1856.

His early education was acquired in the private schools of Brooklyn, N. Y., and the Weston Military Academy of Weston, Conn. In 1873 he was enrolled in the Sheffield Scientific School of Yale College, in New Haven, from which he was graduated with the Class of 1877.

After graduation from college he was employed in the machine shop of the Continental Works, in Brooklyn, which his father, who was sole proprietor, had established in 1859. He was soon assigned to the engineering staff and in 1887 assisted in the incorporation of that establishment under the name of The Continental Iron Works, becoming its first secretary and treasurer, a position that he held until his election to the presidency of the company in 1909. He continued to serve as president until the company was dissolved in 1927, when he retired from active business.

In business life he was quiet and unostentatious, yet contributed a vast amount of assistance to the successful operation of the company both in the financial department and in the solving of engineering problems. He was beloved of all with whom he came in contact.

Mr. Rowland was a member of the American Society of Mechanical Engineers, the Society of Naval Architects and Marine Engineers, the Society of

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[1] Memoir prepared by George A. Tibbals, M. Am. Soc. C. E.

[2] For memoir, see *Transactions*, Am. Soc. C. E., Vol. LXII, March, 1909, p. 547.

Colonial Wars, Sons of the Revolution, the Union League Club of New York, and the Larchmont Yacht Club. He was a trustee of many years' standing of the Webb Institute of Naval Architecture. For many years he served as Junior Warden of Christ Church (Bedford Avenue), in Brooklyn, and was a member of St. Bartholomew's Protestant Episcopal Church of Brooklyn at the time of his death.

He was married in 1880 to Estelle M. Hull and is survived by a daughter, Jessie Rowland (Mrs. William E. Coykendall), and a grandson, William E. Coykendall, Jr.

Mr. Rowland was elected a Member of the American Society of Civil Engineers on September 1, 1886.

ROY FRANCIS RUDOLPH, M. Am. Soc. C. E.[1]

DIED SEPTEMBER 16, 1940

Roy Francis Rudolph was a son of Charles and May (McGregor) Rudolph. He was born in Saint Jo, Montague County, Tex., on November 25, 1885, and spent his boyhood in this and other sections of Texas. He received his early education in the schools of Texas, and in 1911 was graduated from the Agricultural and Mechanical College of Texas, at College Station, Tex., with the degree of Bachelor of Science in Civil Engineering.

In July, 1911, he was employed by the Vicksburg District of the Mississippi River Commission, where he served in various capacities on channel and levee protection work until May, 1917, when he gave up his position as superintendent of construction to enter the United States Army as Captain in the Field Artillery Corps. He served as Battery Commander in the 335th Field Artillery of the American Expeditionary Forces in France until he was returned to the United States and given his honorable discharge at Camp Dix, New Jersey, on March 19, 1919. During the ensuing five years he held a Captain's commission in the Organized Reserve Corps.

In May, 1919, he entered the services of the Mississippi State Highway Commission, at Jackson, Miss., as resident engineer on construction and was later promoted to district engineer of the Second District where he was in charge of federal and state aid road projects. He continued in this position until September, 1926, when he resigned to become the executive engineer for Nixon and Phillips, General Contractors, of Birmingham, Ala. He continued with this firm on highway, street, sewer, and railway work until June, 1928, when he accepted the position of supervising engineer for road work in Union County, Mississippi, where an extensive road improvement program was in progress.

In November, 1932, he again entered the service of the government on river and harbor work in the Montgomery, Ala., Engineer District. He continued in the service of the Army Engineers, in the Montgomery and Mobile (Ala.) districts, until his death on September 16, 1940, at which time he was an associate engineer in charge of the river and harbor improvement work in the Third Field Area of the Mobile District with headquarters at Panama City, Fla.

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[1] Memoir prepared by Truman A. Smith, M. Am. Soc. C. E.

Mr. Rudolph was married to Mary Pendleton on June 18, 1914, at Stratford, Tex. He is survived by his widow, one daughter, Voncile (Mrs. C. W. Kitchens), and one son, Dwight Pendleton.

Mr. Rudolph will be remembered by those who were privileged to know him for his ability as an engineer, and for his strong and jovial personality. He possessed a keen sense of humor and ready wit, and made work and play boon companions.

He was a member of the Society of American Military Engineers.

Mr. Rudolph was elected an Associate Member of the American Society of Civil Engineers on July 6, 1925, and a Member on January 26, 1931.

JOSEPH ANDREWS SARGENT, M. Am. Soc. C. E.[1]

Died April 20, 1939

Joseph Andrews Sargent was born on September 29, 1872, in Escanaba, Mich., the son of Charles Henry Sargent and Mary Olive (Andrews) Sargent, natives of Gloucester, Mass. They were of English ancestry.

His father, a civil engineer, was one of the early railroad builders of the West, whose work extended to various parts of Western United States. The early education of the son Joseph was limited to the district school, a mile and a half from the farm at Garrison, Nebr., where the family was living during the decade following 1881. Later there was school in Lincoln, Nebr., alternating with periods on various work. In 1891 he went with his father on some railroad construction work in Mississippi, and it then seemed inevitable that his own life work should be engineering.

Mr. Sargent entered the University of Nebraska, at Lincoln, in 1894. The following years seem to have been difficult in Nebraska, as elsewhere. The family had suffered financially in the panic of 1893, and it was necessary for him to contribute a major part of the cost of his four years at the university. He arranged this by beginning his school year late and ending it early, leaving long summers for such work as might be available, usually on railroad location or construction. Those were railroad building years, and this was the work most readily available for young engineers. Because of his absence in Cuba in 1898, he failed to graduate with his class, but received the degree of Bachelor of Science in Civil Engineering in 1903.

In April, 1898, he enlisted in Company H, 2d U. S. Volunteer Engineers, was appointed sergeant and served at Camp Mead, in Middletown, Pa., where he worked on the camp water supply. When the protocol with Spain was signed in August, 1898, the regiment was still in the United States, but was later sent to Cuba, reaching Habana in January, 1899, where it was employed mainly on much-needed sanitary work.

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[1] Memoir prepared by George C. Danforth, M. Am. Soc. C. E.

Mr. Sargent was mustered out in Habana and became a civilian employee of the Corps of Engineers, U. S. Army, under Col. (later Maj. Gen.) William M. Black,[2] M. Am. Soc. C. E., and in February, 1900, was placed in charge of the "Survey of Fortifications." The work included the establishment of a base line for fire control for the harbor forts and the development of a triangulation system in and around the City of Habana. When the Cubans took over the government in 1902, most of the American employees, including Mr. Sargent, were dismissed.

He spent the summer of 1902 as resident engineer on a cutoff under construction for the Mexican Central Railroad Company. In November, 1902, he returned to the United States to join the newly organized Reclamation Service, in which he served as principal assistant to the project engineer on the design of the irrigation system and main canal of the Uncompahgre Project in Colorado, and the six miles of location surveys for the Gunnison Tunnel. Later he was in charge of engineering on the Gunnison Tunnel construction. The intense interest with which he attacked the problems of the day's work, and his helpful attitude toward younger engineers, were evident on this work, and must have contributed toward the esprit de corps of an organization composed largely of young engineers, many of whom later became distinguished members of the profession.

Late in 1906, after six months on the East River tunnel construction between Manhattan and Brooklyn, N. Y., he returned to Cuba, serving for a year as chief engineer of the Cuba Railroad Company. This work was well under way when it was stopped by the revolution of 1906. During the "2d Intervention" (the Second U. S. Provisional Government of Cuba) in 1907 and 1908, American engineers were used on much needed main-line highways and bridges. Mr. Sargent was in charge of a section from Manzanillo, on the southeast coast, to Bagamo, some fifty miles north. The work was turned over to the Cubans in 1908, and he returned to the United States.

The years 1909 to 1913 were given to a general engineering practice, including irrigation and railroad projects in Western United States, followed by an engagement on the design of the Medina Dam[3] or an irrigation development some thirty miles southwest of San Antonio, Tex. In 1913 he was sent to Spain by the F. S. Pearson Syndicate in connection with a hydroelectric development for the Barcelona area. He served as chief engineer of the Riegos y Fuerza del Ebro, S. A. (société anonyme),[4] taking charge of construction at Seros and Camarase on the Rio Segre and at Tremp on the Rio Noguera Pallaresa. The Seros development included the largest dam in

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[2] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 99 (1934), p. 1411.

[3] "Design and Construction of Dams," by the late Edward Wegmann, M. Am. Soc. C. E., New York, N. Y., John Wiley & Sons, Inc., 1927.

[4] "Seros Power Development, Lerida, Spain: Diversion Dam and Canals," by J. C. Stevens, M. Am. Soc. C. E., *Engineering News*, September 3, 1914, p. 473; also "The Seros Hydro-Electric Development Near Lerida, Spain," by the same author, *loc. cit.*, September 10, 1914, p. 536.

Europe (at that time) with a long, concrete-lined power canal. This 40,000-kw project resulted in a considerable growth of the population and business of Barcelona, and its capture by the revolutionists had an appreciable influence on the collapse of governmental resistance there.

Mr. Sargent went from Spain to Paris, France, in 1917, and soon after the entrance of the United States into the war was given a commission as Captain of Engineers and assigned to railway yard location and construction for the supply depot at Gievres. His experience had provided an unusual equipment for the engineering problems of the Service of Supply, but he was insistent on service with combat troops and was assigned to the 2d Engineers with the 2d Division. He was severely wounded by a shell splinter at Lucy-le-Bocage, near Château-Thierry, on June 2, 1918. After a period in a military hospital, he was returned to duty on August 10, and rejoined his regiment in time to take part in the offensive of the First American Army at Saint-Mihiel.

During the Meuse-Argonne offensive, while the division was serving with the French Fourth Army of General Gouraud, Captain Sargent carried out a reconnaissance of enemy positions on October 6, 1918, for which he received the following citation and the Croix de Guerre:

> "Le Maréchal de France
>
> "Commandant en Chef les Armées Françaises de l'Est, après approbation de Monsieur le Général Commandant en Chef les Forces Expéditionnaires Américaines en France, cite à l'ordre de la Division: No. 13.372 "D" Capitaine J. A. Sargent, E.M. du 2 Régiment de Génie Américain:
>
> "'Le 6 Octobre 1918, près de St-Etienne-à-Arnes, s'est exposé avec sang-froid et courage, sous un violent feu d'Artillerie et de mitrailleuses en faisant une reconnaissance en première ligne. Est revenu avec des renseignements utiles et très précis.'
>
> Au Grand Quartier Général, le 10 Février 1919.
> Le Maréchal Commandant en Chef
> Les Armées Françaises de l'Est,
> (sgd) Petain"

Free translation:

> "The Marshal of France
>
> "Commander in Chief of the French Armies of the East, after approval of the Commander in Chief of the American Expeditionary Forces in France, cited at the order of the Division: No. 13.372 "D" Captain J. A. Sargent, of the staff of the 2d Regiment of American Engineers:
>
> "'The sixth of October, 1918, near St. Etienne-à-Arnes, he faced danger with coolness and courage, under a violent Artillery and machine gun fire, in making a reconnaissance in the first line. He returned with useful and very exact information.'
>
> At General Headquarters, February 10, 1919.
> The Commander in Chief
> The French Armies of the East,
> (signed) Petain"

It may be noted that the 2d Engineers had a somewhat unusual record for use as infantry, much of their engineering work being done at night.[5]

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[5] Historical Report of the Chief Engineer, Am. Expeditionary Forces, 1919, p. 175 *et seq.*

Shortly before the Armistice of November 11, 1918, he was assigned to the construction of base hospitals in the south of England, later being sent to Chaumont, France, as post engineer, where he was discharged from the service in the spring of 1919. Captain Sargent rarely talked of his war experiences, even with friends who had also served in engineer regiments of the American Expeditionary Forces, and the writer recalls only his apparent satisfaction at having known the commanding general when the latter was instructor of cadets at the University of Nebraska.

Following his discharge from the army he was employed in a consulting capacity on various regional developments in Spain and Poland under the Paris office of the Foundation Company of New York, and returned to New York in 1920.

In the latter part of 1920, D. P. Robinson and Company, Inc., obtained a contract with the Brazilian Government for the construction of five dams in the states of Ceará and Parahiba, in connection with work proposed for the rehabilitation of the so-called "Dry Northeast." Captain Sargent was sent down in August, 1921, nominally as superintendent of one of the dams. This was due to an agreement as to the number, titles, and salaries of imported men. The late I. W. McConnell,[6] M. Am. Soc. C. E., executive vice-president of the company, stated that Mr. Sargent's experience and clear thinking were not to be wasted in superintending a construction job. He was wanted as a consultant on all the work and was so used. Although the work extended over three years, none of the dams was completed, because of the failure of the government to arrange the necessary financing. Captain Sargent was later made general superintendent and had the difficult task of the final cleanup after the work was suspended early in 1923.

In 1924 and 1925 he made some investigations for the same company in Poland, and in 1926 and 1927 had charge of irrigation exploration for the J. G. White Corporation in the state of Durango, Mexico, covering the Guatimapé Project and various dam sites.

Early in 1928 he joined the Emprezas Electricas Brasilieras, S. A., the Brazilian subsidiary of the Electric Bond and Share Company, and was sent to the State of Paraná in Southern Brazil, where the most important project in hand was the location of a hydroelectric station to supply power for the City of Curityba. A number of possible sites were available on small rivers of variable flow near the crest of the coastal range, the Serra do Mar. There were no records of flow, no adequate maps, few records of the heavy rainfall, and no roads in the heavily forested area. His work was usually in the saddle or on foot, supervising reconnaissance parties and weighing various conditions of available head and flow, power canals, and tunnels. The final choice of a site on the Rio São João, later known as Chamine, required a knowledge of the data essential to his analysis and ability to evaluate these data accurately and quickly.

On completion of the work in Paraná, he spent several months at Bananeiras in the State of Bahia as consultant on the concrete dam under construction there, and was there during the revolution of 1930. The depression put a halt to the South American work, and he returned to New York in 1930, taking an apartment at Jackson Heights, Long Island, N. Y.

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[6] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 101 (1936), p. 1592.

After a period of inactivity, he found an opportunity for interesting work on the new Long Island subways and, during the summer of 1934, was appointed resident engineer and assigned to the construction of the Eighth Avenue section of the Independent Subway System. In the fall of 1936 he was promoted to supervising engineer, Public Works Administration (PWA), in charge of that work and, soon after, was transferred to the Lincoln Tunnel crossing of the Hudson River, where he was employed until shortly before his death.

It is difficult to summarize such a career. Of high professional ability and with an unusual degree of intellectual honesty, Captain Sargent never lost sight of the humanities, and could bring the same untiring energy to the construction of a dam or the exploration of a stream or lake for trout or salmon. He had more than his share of humility and kindliness and a rare appreciation of the qualities of his fellow workers and friends. If he saw faults in them, he was never known to mention them to others.

With a long and varied experience and a retentive memory, he was especially valuable in reporting on various phases of construction programs and in the comparing of alternative projects, where a thorough analysis of detail was essential. He was the product of an era that may have ended during the years following the war period of 1914–1918, when the use of the laboratory was replacing to some extent the developed judgment based on a broad knowledge of fundamentals. It is believed that in his passing, with others of his type, the engineering profession has sustained a loss that is not being—perhaps cannot be—replaced.

He was married at Habana in June, 1900, to Mary A. Horne of Lincoln. She died in 1925 and in 1928 he was married to Barbara C. Ripley, who survives him. He had two sons by his first wife, Henry Alfred Sargent, Assoc. M. Am. Soc. C. E., and William Horne Winthrop Sargent. William died in 1938.

Mr. Sargent was elected an Associate Member of the American Society of Civil Engineers on October 2, 1901, and a Member on April 21, 1930.

FRANCIS CLINTON SHENEHON, M. Am. Soc. C. E.[1]

DIED OCTOBER 3, 1939

With the passing of Francis Clinton Shenehon, the profession lost an outstanding personality. He was a man of wide interests, and a conspicuous figure in every gathering. He was conversant with writings on all phases of human life. His long and successful engineering career acquainted him with all parts of the United States and took him twice to Europe. He counted among his friends and personal acquaintances a large group of engineers whose works long have ranked as classic accomplishments to younger men. Although of dignified personality and rather formal demeanor, he often turned quickly from the serious aspects of a problem to a funny story to illustrate a point, or for mere relaxation.

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[1] Memoir prepared by Adolph F. Meyer, M. Am. Soc. C. E.

"Dean" Shenehon (as he was generally known after serving the University of Minnesota in that capacity) took great pride in his profession and its possibilities for service. He was a "prime mover" in the formation of the Minnesota Joint Engineering Board, predecessor to the Minnesota Federation of Architectural and Engineering Societies, and was a zealous attendant at local and national engineering meetings and conventions.

When the Society met in Minneapolis, Minn., in June, 1919, for its Forty-Ninth Annual Convention, Dean Shenehon made the outstanding address. Subsequently, an editorial in the technical press[2] urged every engineer "to read with care and reread this address," and closed with the comment:

> "It is not merely interesting and informative, but is an unusual model of something we sadly lack in our engineering writing—a technical theme handled with grace and charm, losing nothing of its technical value, but rather being enhanced by the treatment."

The closing paragraphs of that address of Dean Shenehon well illustrate how beautifully, yet emphatically, engineering facts may be stated:[3]

> "* * * Budgeting the water means spending it systematically and scientifically, in well-thought-out apportionments, which will make the Niagara and the St. Lawrence better navigable ways and better water-power streams.
>
> "Water power is really the thing which inflames the imagination. Aladdin with his lamp never conjured to serve him genii so omnipotent as engineers have made emerge from seaward bound waters; and the Niagara and the St. Lawrence descending to the ocean hold in their depths potentialities to assume tremendous burdens of labor, which still pitilessly strain human muscles.
>
> "These things of which I have spoken I conceive to be, in the largest sense, the significant Engineering Activities of the Twin Cities."

Dean Shenehon set an example for all engineers in the manner in which he mingled with men in all professions and businesses and in the active part he took in the life and activities of the community, wherever he lived.

Francis C. Shenehon's contributions, during his long engineering career, grew out of individual accomplishments rather than out of leadership of groups of men. His mind was a reservoir of original ideas. The boldness of his conceptions at times startled even those of young and daring years. He looked upon every project and every problem as a challenge to engineering ingenuity and a mandate to search for new and better solutions. He loathed pettiness and red tape and delighted in taking direct paths to his goal, even though his footsteps led over others' toes. He was lavish in his praise of those in whom he had confidence, even to their own frequent embarrassment. His condemnation of professional weakness was equally frank.

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[2] *Engineering News-Record*, June 26, 1919.

[3] *Proceedings*, Am. Soc. C. E., August, 1919, p. 588.

Conspicuous among Mr. Shenehon's accomplishments was the invention of the "long wire sweep" by means of which countless uncharted pinnacle rocks and wrecks have been located in even the most thoroughly sounded navigable waters. The idea has been adapted and utilized in mine sweeping operations. Life and property owe much to that one brilliant conception.

While engaged on the problem of preserving the grandeur of Niagara Falls by limiting diversion and preventing the cataract from destroying itself by erosion, Mr. Shenehon developed the technique of gaging large streams to new and precise standards that have permitted accurate computations of the outflow from the Great Lakes and a better determination of the supply factors which are fundamental to the regulation of those lakes and the preservation of the Falls.

Francis Clinton Shenehon was born in Brooklyn, N. Y., on December 20, 1861. His mother was Ella (Dalley) Shenehon. His father, Ellsworth S. Shenehon, had a large wholesale mercantile establishment on Broadway in New York, N. Y., until Francis was seventeen years old and the family moved west. The young man wanted to become a lawyer and, for some time, studied in the office of a Minneapolis attorney. It is a significant fact that, during his later years, a large part of Mr. Shenehon's engineering work was associated with legal proceedings.

Young Shenehon's health was not good, so when an opportunity offered, he joined a survey crew of the Minneapolis, St. Paul and Sault Ste. Marie Railway Company and went into camp. He enjoyed this work so much that he decided to become an engineer instead of a lawyer, and in 1881 enrolled in the College of Engineering of the University of Minnesota, at Minneapolis. After three years of work in college, his health again made it desirable to do outdoor work, so he re-entered the employ of the "Soo Line." There he gained experience on surveys, bridge construction, track laying, and the like. From 1888 to 1890, he was a member of the firm of Cox and Shenehon, engaged in general engineering practice at Sault Ste. Marie, Mich., including surveys, sewer, water supply, bridge designs, and supervision of construction.

Mr. Shenehon next entered the employ of the Federal Government in the War Department, on the construction of New Fort Brady at Sault Ste. Marie, and on the 800-ft ship lock in the St. Marys Canal. Besides instrument work, he gained experience on masonry construction, channel excavation, and winter earth work. During this period, on May 14, 1891, he was married to Kate Bird Cross.

From 1893 on, he was assigned to responsible work in connection with the lock and power-house construction at the "Soo," and also was engaged on latitude and longitude observations of the U. S. Lake Survey, the making of discharge measurements of St. Marys River, steel and bronze wire base-line work, and then the making of the Mackinaw (Mich.) base-line reductions.

During this period he continued with his college work by correspondence, and in 1895 he was granted the degree of Bachelor of Science in Civil Engineering by the University of Minnesota. In 1900 he received the degree of Civil Engineer from the same university.

From 1898 to 1909, Mr. Shenehon was on the U. S. Lake Survey, first in Buffalo, N. Y., then in Ogdensburg, N. Y., and later in Detroit, Mich. He was engaged in hydrological and hydraulic work on the St. Lawrence, the Niagara, the Detroit, and the St. Clair rivers.

It was while working on the Great Lakes and their connecting rivers that Mr. Shenehon invented the "long wire sweep" and developed more precise methods of making stream gagings. Between 1906 and 1908, he gathered the data that formed the basis for his comprehensive report on "The Preservation of Niagara Falls."[4]

In 1909 Mr. Shenehon was called to his alma mater as Dean of the College of Engineering. While serving in this capacity, Dean Shenehon emphasized the need for the training of high-grade engineers and the raising of the standards of the engineering profession. He made every effort to strengthen the teaching personnel and to make student training more effective. He urged the engagement of professors on outside work in their own specialty, but frowned upon routine practice and the solicitation of competitive work. He introduced a new course of lectures and inspections given to freshmen by the highest ranking professors in order to help the freshmen get oriented and to guide them in selecting the most suitable field of engineering in their sophomore year.

While at the University, Dean Shenehon carried on considerable private practice. He served as expert for the United States in the celebrated Chandler-Dunbar case, involving the condemnation of water power at Sault Ste. Marie. He was engaged by the Hydraulic Power Company in connection with power development at Niagara Falls. He served as expert for the United States in the long-drawn-out litigation against the diversion of water from Lake Michigan by the Sanitary District of Chicago, Ill. The record of this case constitutes almost a "three-foot bookshelf" in itself.

In 1918, shortly after Dean Shenehon had resigned his position to devote all of his time to private practice, he was employed by the Sanitary District of Chicago to report on remedial measures and contributed a comprehensive report[5] on the "Regulation of the Niagara and St. Lawrence Rivers," which embodied novel control works.

For a brief period, beginning in 1919, Francis C. Shenehon and the writer practised as partners in the firm of Shenehon and Meyer. It soon developed, however, that it was more advantageous for them to practise independently. Until 1923, when Dean Shenehon was elected vice-president of Byllesby Engineering and Management Corporation, they maintained joint offices and staff. During this time, with the full approval of their respective clients, they were engaged as experts on opposite sides of litigation before the United States Supreme Court.

Among the other projects on which Dean Shenehon was retained were the following:

The protection of the water supply at Natchez, Miss.
The Minnesota-Dakota "Million Dollar Drainage Case."

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[4] Report of Chief of Engineers, U. S. Army, 1908.

[5] Report, Sanitary District of Chicago, 1919.

The Sioux Falls (S. Dak.) spillway—a unique structure in which he caused the flow of a large drainage ditch to be dropped down a vertical shaft about 100 ft, then turned into the horizontal, carried out to the edge of the bluff on solid rock, and turned downward into a stilling pool to destroy the energy.

The repair of the Coon Rapids dam over the Mississippi River, in Minnesota, by lowering the apron and constructing a stilling pool.

The development of water power at Marseilles, Ill., on the Illinois River.

The development of water power at Louisville, Ky.

During the condemnation of riparian lands and improvements along the Mississippi River, growing out of the construction of the 9-ft channel, Mr. Shenehon spent many weeks on the witness stand as engineer for the Milwaukee and the Burlington railroad companies. He had the knack of stating engineering principles in simple terms and expressing complicated hydraulic engineering relationships in homely phrases and with the aid of bold and simple drawings and models. Juries listened intently and gave much weight to his evidence.

In 1924 Dean Shenehon presented a comprehensive paper before the Society entitled "The St. Lawrence Waterway to the Sea."[6] From 1924 to 1929 he was chairman of the Society's committee on "Ice as Affecting Power Plants."

He was a member of the American Institute of Consulting Engineers, the Engineers' Club of Minneapolis, the Minnekahda Club, the Minneapolis Club, the Interprofessional Institute, the Six O'Clock Club, the scholarship fraternities Tau Beta Pi and Sigma Xi, and of Delta Tau Delta and Alpha Kappa Sigma.

During his last years and after his second European trip in 1929, Dean Shenehon spent portions of many winters in the South and summers on the shore of Lake Superior as a member of the exclusive Encampment Forest Association.

Dean Shenehon is survived by three daughters, Eleanor Norcross Shenehon, Clare (Mrs. Fiske Boyd), and Katherine (Mrs. Lewis Child). His widow survived him only seven weeks. He is also survived by his sister, Mary (Mrs. Arthur C. Loring), and two grandchildren, Sheila Shenehon Boyd and Francis Shenehon Child.

Dean Shenehon was elected a Member of the American Society of Civil Engineers on May 7, 1902.

CHRISTOPHER ELIAS SHERMAN, M. Am. Soc. C. E.[1]

DIED MAY 6, 1940

Christopher Elias Sherman, the son of Sylvester Morrill Sherman, M.D., and Lemyra Ann (Shoemaker) Sherman, was born in Columbus, Ohio, on December 28, 1869. He was educated in the schools of Columbus and, after three years in field and office work with the county surveyor, was enrolled in the Ohio State University, at Columbus, from which he was graduated with the degree of Civil Engineer in 1894.

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[6] *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 444.

[1] Memoir prepared by Clyde T. Morris, M. Am. Soc. C. E.

After two years in engineering work in the West he returned to his alma mater as assistant in the Civil Engineering Department, and advanced through the grades of assistant professor and associate professor to professor of Civil Engineering and head of the department, to which position he succeeded upon the death of Prof. C. N. Brown in 1902. He held this latter position until 1938 when he resigned the chairmanship of the department but continued his teaching duties until his retirement in September, 1939.

Professor Sherman was always active in public affairs, especially in the field of flood control, stream flow, and water conservation. He served the State of Ohio as the governor's representative on the Ohio Cooperative Topographic Survey from 1902 until his death, and under his direction Ohio was the first state in the Union to be completely mapped. He was chief engineer and later chairman of the Scioto Sandusky Conservancy District, and took a leading part in the cooperative work of the U. S. Geological Survey, Water Resources Branch in Ohio.

He prepared and published the preliminary report on the Ohio Topographic Survey in 1904, progress report of the Ohio Topographic Survey in 1910, "The Ohio Water Problem" in 1915, and the final report of the Ohio Topographic Survey, Volume I in 1916, Volume II in 1917, Volume III in 1925, and Volume IV in 1933. He also wrote the book entitled "Ohio Stream Flow" in 1932.

Professor Sherman was affiliated with the following organizations: Sigma Xi, Sigma Nu, and Tau Beta Pi fraternities, Triangle, Kit-Kat, Torch, Optimist, and Faculty clubs. He served as president of the Ohio Engineering Society for the year 1913–1914. He was a life-long member of the First Congregational Church in Columbus and took an active part in its work.

He was married to Elenorea Bruning on June 22, 1897, who survives him. To this union were born three sons, Robert Bruning Sherman, Theodore Bruning Sherman, and Henry Bruning Sherman, who also survive him.

Aside from his engineering pursuits, Professor Sherman was a student of literature, history, and biography, especially the history of the United States and its great personalities. He was also a writer of delightful prose and poetry. In 1936 he published a small volume of verse and prose entitled "The Land of Kingdom Come," which he sent to each of his former students (numbering about a thousand) as a memento of his friendship and interest. None of his former students ever passed in the vicinity of the University without dropping in for reminiscences and a chat. His genial nature and lovable personality may be judged from the following quotation from the preface to "The Land of Kingdom Come":

"Come With Me
"And I shall lead in far fields with that gayest
of companies—college students and surveying
crews. If you will join us, we shall climb rugged
peaks, loiter along clear streams, or travel
the dusty plain; for the surveyor surveys these—
and canyons grand and imposing!

"Does your mood demand sober facts? You shall
have them in plain prose tales, for all but the
shortest of those that follow are true. Or if
you like, we shall walk on the edge of the
sunset, into the borderland between fact and
fancy, and take flight on the wings of verse.

"If I steal upon your mellow mood around the
corner of an unsuspected paragraph, or surprise
your lurking fancy at a page here or there—
be it only a page now and then—why what more
can one ask of a little book such as I, for
casual reading only."

Professor Sherman was elected a Member of the American Society of Civil Engineers on June 6, 1927.

RAPHAEL JOSEPH SMYTH, M. Am. Soc. C. E.[1]

Died September 15, 1939

Raphael Joseph Smyth was born in New York, N. Y., on September 16, 1879, the son of Michael Gilbert Smyth and Mary Josephine (Carroll) Smyth. His early education was acquired in the grade schools of that city. Having decided upon an engineering career, he pursued studies at Cooper Union, in New York City, where he received degrees in science and engineering. This training was supplemented by further work in engineering and courses in architecture and law at Brooklyn (N. Y.) Polytechnic Institute; at Columbia University and New York University, both in New York City; and at home. Filled with a thirst for knowledge and a love of learning that led his thoughts into many fields of mental endeavor, he remained a student throughout his life.

Mr. Smyth began his engineering career in 1904 as a rodman in the Rapid Transit Commission of the City of New York. After holding successively the positions of draftsman and assistant engineer with that agency, he accepted an appointment in 1907 as a transitman and computer in the office of the President of the Borough of the Bronx. In the years that followed his first association with the Bronx, he was in turn assistant engineer, assistant engineer of design, engineer of design, deputy chief engineer and, at the time of his death, chief engineer of that borough of 1,500,000 people. During this period, he was identified in some measure with the design or construction of practically all of the improvements coming within the purview of the office of Public Works in the Bronx, involving an expenditure in excess of $200,000,000. He was recognized as an able designer in the fields of steel and reinforced concrete and introduced various innovations in connection with the design of sewers, highways, buildings, bridges, arches, and other types of structures. As chairman of a special committee appointed to revise specifications and contracts for highway work in the City of New York, he devoted himself so assiduously to his task that his health was seriously impaired even before the completion of the work, which required several years for fulfilment. When he took over his duties as chief engineer, in January, 1938, the burden of this added responsibility began to take its toll and, despite heroic efforts to conceal the fact, his health declined rapidly, culminating in his untimely death on September 15, 1939.

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[1] Memoir prepared by Arthur V. Sheridan, Assoc. M. Am. Soc. C. E.

In addition to his career in the public service, Mr. Smyth had for some years pursued a private practice as an architect, engineer, and surveyor, and during the World War of 1917 he was engaged in government work on military and naval construction. He was an architect and engineer, having been legally admitted to the practice of both professions in the state of New York.

Extremely fond of, and conversant with, good music, he had, as a young man, learned to sing and to play the violin. His love of the lyrical, supplemented by his store of knowledge and wide background of reading, made him a welcome companion in any company. Possessed of poise and erudition, he suggested the educated man in speech and deportment. He was an ardent advocate of the opinion that a truly educated man is the product of his own efforts, not the result of a school, and he labored unceasingly to promote for every man an opportunity to demonstrate his knowledge and talents, regardless of where or how acquired. He firmly believed that such a road to achievement was in the best interests of society, profession, and individual.

Active in cultural, social, and civic as well as professional affairs, Mr. Smyth was at various times a member of a literary club, a debating society, a chamber of commerce, the Bronx Board of trade, the Bronx Real Estate Board, architectural groups, engineering societies, and political organizations. He was a pioneer in movements that envisioned a better future for the engineer and gave freely of his talents and finances to promote such an end. He was one of the founders of the New York State Society of Professional Engineers and was a past-president of its Bronx County Chapter, as well as its first official historian. He was also a past-president of the Municipal Engineers Society of New York, a member of the National Society of Professional Engineers, and a member of the Bronx Council 266, Knights of Columbus.

On October 9, 1906, he was married to Agnes F. Reilly, who was his constant and loyal partner throughout the years that followed. Mrs. Smyth survives him.

Mr. Smyth was elected a Member of the American Society of Civil Engineers on October 10, 1921. He represented in full measure the high ideals and standards of membership in that organization.

ROY MARTIN SNELL, M. Am. Soc. C. E.[1]

DIED JUNE 29, 1941

Roy Martin Snell was born in La Grange, Me., on August 14, 1880, the son of Willard and Ida W. (Cushman) Snell. His ancestors were among the first to settle in the Thirteen Colonies.

Mr. Snell attended Bridgton Academy, at North Bridgton, Me., and was graduated in 1900. While there, he was elected to Alpha Chi Sigma Fraternity. He was graduated from the University of Maine, at Orono, Me., in 1905 with a degree of Bachelor of Science, having taken an engineering course. He was a member of the Phi Kappa Sigma Fraternity.

Following his graduation, he was employed for about one year by the Maine Central Railroad Company as a rodman and inspector of bridge abutment construction.

Believing that there was a larger field available with the United States Reclamation Service, Mr. Snell left the railroad and in May, 1906, joined the Service as a field assistant. From then until March, 1924, he served continuously with the Reclamation Service (with the exception of one year as superintendent of construction in charge of the building of a concrete canal syphon under the Laramie River in Wyoming). His activities while with the Reclamation Service covered a large variety of construction work in Montana and Wyoming on the Huntley, Shoshone, Flathead, Blackfoot, Sun, and Milk River projects, which included the building of dams, canals, and tunnels. Mr. Snell's ability as an engineer and an executive were recognized by the government, and he was progressively promoted through various positions to that of project engineer.

In 1924 the government of Puerto Rico requested the chief engineer of the U. S. Reclamation Service to recommend a competent engineer to take charge of the construction of the Guajataca Dam, a 120-ft high, semi-hydraulic fill structure, together with a 14,000-acre distribution system. Mr. Snell was selected and granted a furlough from the Reclamation Service, and in March, 1924, proceeded to Puerto Rico with the expectation that he would remain there possibly three years. However, before this initial work was completed, other projects were in the making, and Mr. Snell remained in Puerto Rico until November, 1934, building, in all, three dams, a power canal, and various appurtenant structures.

In October, 1935, Mr. Snell returned to the U. S. Bureau of Reclamation and was sent to Redding, Calif., where surveys were being started for the Shasta Dam and the relocation of some thirty miles of new railroad to replace an existing portion of the Southern Pacific, part of which would be submerged by the reservoir to be created by the construction of Shasta Dam on the upper Sacramento River. Mr. Snell organized and supervised these surveys, and directed the construction of Government Town located some ten miles north of Redding. He later had charge of the construction of the new railroad, which was a work of the first magnitude, involving the construction of twelve tunnels with an aggregate length of 19,070 ft, eight steel bridges totaling 12,202 ft, and 5,800,000 cu yd of grading. During his last illness he completed an article for *Civil Engineering*[2] describing in detail the construction of the largest bridge on the project, the double-deck structure crossing the Pit River. With this work drawing to a close, it seems a queer trick of fate that did not permit Mr. Snell to see the completion of a project in which he was so interested.

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[1] Memoir prepared by J. A. Given, M. Am. Soc. C. E.

On October 11, 1902, Mr. Snell became a member of Composite Lodge No. 168, F. and A. M., of La Grange, and throughout the remainder of his life he was very much interested in Masonic affairs.

He possessed a high sense of duty and exemplified in his own life and conduct the highest ideals of an engineer. He was always courteous and gentlemanly and took the greatest interest in his family. He spent several years in tracing the genealogy of the Snell family to pass on to his three sons.

He was married on January 1, 1914, in Helena, Mont., to Olivia E. Connor, who, with three sons, Willard J.; Roy M., Jr.; and James A., survives him. He also leaves one sister, Mrs. Evelyn Snell Abbott.

Mr. Snell was elected an Associate Member of the American Society of Civil Engineers on September 6, 1910, and a Member on June 20, 1922.

VERNON LYLE SULLIVAN, M. Am. Soc. C. E.[1]

Died July 12, 1938

Vernon Lyle Sullivan, the son of George W. and Mary A. (Beeman) Sullivan, was born in Roxana, Mich., on May 23, 1876. He received his elementary education in the village school of Roxana. Later he attended high school in the nearby town of Charlotte, Mich. While in his early twenties, he moved with his parents to Grand Ledge, Mich., and from 1898 to 1902 he studied engineering under the private tutorage of the late Prof. W. D. Southworth of Grand Ledge.

In 1902 his health failed, and he was told that he had but a short time to live. Displaying then his most characteristic trait of refusing to accept defeat even in the face of overwhelming odds, he proceeded to outfit himself with specially designed camping equipment, left his home, relatives, and friends, and journeyed to the Pecos Valley in New Mexico. There he made camp near the town of Carlsbad, N. Mex., where he lived outdoors—hunting, fishing, and pursuing his studies until he regained his health.

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[2] *Civil Engineering*, October, 1941.

[1] Memoir prepared by John J. Ledbetter, Jr., M. Am. Soc. C. E.

In 1903 Mr. Sullivan accepted the position of chief engineer for the Pecos Irrigation Company, which at that time owned and operated the largest irrigation system in the southwest. He served in that capacity until the Irrigation Company sold the system, later known as the Carlsbad United States Reclamation Project, to the United States Government in 1904. He remained on the Carlsbad Project with the U. S. Bureau of Reclamation for a short time, and then entered private practice in Carlsbad.

During 1905 and 1906 he designed and constructed the Public Utilities Hydro-Electric Power Plant at Carlsbad, and the Carlsbad Power and Light Hydro-Electric Plant. He also designed and installed the water works system for the City of Carlsbad and many other smaller projects.

Mr. Sullivan was appointed territorial engineer of the territory of New Mexico in March, 1907, by Governor Herbert J. Hagerman and continued to serve in that capacity under Governors George Curry and William J. Mills. While serving as territorial engineer, he had charge of all water appropriations in the state and in that capacity worked untiringly for the Elephant Butte Project (later the Rio Grande Federal Irrigation Project), making one trip to Washington, D. C., and several trips to the proposed dam site with members of Congress and others who were investigating the proposed project. He loyally protected the government's water filings for the project, which has been, and will continue to be, of the greatest benefit to El Paso, Tex., and to the Rio Grande valleys in Texas and in New Mexico. While serving as territorial engineer, he also had charge of the construction of all New Mexico state highways. Among the principal highways built under his direction were the Albuquerque-Santa Fe Highway, which had the unique La Bajada Hill switchbacks; the Carrizozo-Roswell State Road; the Raton State Line Road; and El Camino Real.

Mr. Sullivan resigned as territorial engineer for New Mexico on December 31, 1910, in order to accept the position of engineer-manager of the irrigation projects of the Orient Railroad Company in Texas, and from 1911 to 1913 he had charge of the construction, operation, and maintenance of all of the irrigation projects owned and operated by that company.

During the next four years, he designed and constructed irrigation projects in southwestern Texas. The more important of these projects were known then as the Fort Stockton, the Balmorhea, and the Leon Springs Irrigation projects. The Fort Stockton Project was probably one of the best smaller irrigation projects in the Southwest, having concrete-lined canals provided with positive watertight turnout gates and measuring devices.

In 1918 Mr. Sullivan opened an office in El Paso and began private practice as a consulting engineer. From 1918 to 1921 he planned and assisted in financing the Big Wichita Irrigation and City Water Supply Project near Wichita Falls, Tex. Between 1920 and 1933 he served as consulting engineer for the Pecos Valley Water Users Association of Texas; the State of Texas in connection with the New Mexico-Texas Pecos River Water Compact; the Rio Grande Valley Survey Commission of New Mexico; the Basaseachic Hydro-Electric Power Project in western Chihuahua, Mexico; the Bluewater Irrigation District, Bluewater, N. Mex.; and Cía. Agrícola y de Fuerza Eléctrica del Río Conchos, S. A., La Boquilla, Chihuahua, Mexico, in connection with the design and construction of the Colona Power Dam. From 1923 to 1928 he was engineer and appraiser for the Federal Farm Loan Board in the Tenth District of Texas. In 1927 he purchased the El Paso Testing Laboratories in El Paso, and from that year until his death he devoted such time and energies as were not taken up by his engineering practice to the active management and direction of the operation of the laboratories.

During the last seven years of his life Mr. Sullivan devoted the greater part of his time to the Red Bluff Project on the Pecos River in Texas. He made the studies and investigations for the project; designed and supervised the construction of the Red Bluff Dam and Hydro-Electric Plant; and continued as consulting engineer for the Red Bluff Power Control District in connection with the operation of the project.

Mr. Sullivan was a man of many and varied interests and abilities—an experimenter and an inventor. Even as a small boy, he displayed great interest in mechanical improvements, and many amusing incidents arose out of his youthful experiments. On one occasion, he attached a churn filled with cream to a windmill so that the laborious task of producing butter might be more easily accomplished. The pieces of the churn were never found, but his ardor was not dampened for he continued to experiment and in later life invented and patented many useful appliances and devices, including an electrical windmill generating device, a hot-air heating furnace, and an anti-undulating suspension bridge.

Mr. Sullivan was genial and friendly and always had a smile on his face. He was devoted to his family and his church, and served two terms as chairman of the Board of Stewards of the Asbury Methodist Church in El Paso. He was also interested in civic affairs, and served as Trustee of Southwestern University, at Georgetown, Tex. In 1923 he was persuaded to run for Congress as the Republican candidate from the Sixteenth Congressional District of Texas. Although he was not elected, he received the largest vote ever cast for a Republican candidate from that district.

Mr. Sullivan was a member of the American Association of Engineers and served as president of the El Paso Chapter of the Association in 1924 and 1925. He was a member of the American Society for Testing Materials, and was a Thirty-Second Degree Mason.

He was married on June 21, 1904, to Velma A. Holcomb of Carlsbad, who, with three children, George Curry, Marion Adelaide, and Virginia Lou, survives him. He is also survived by a sister, Anna Pearl Sullivan.

Mr. Sullivan was elected an Associate Member of the American Society of Civil Engineers on June 6, 1911, and a Member on September 3, 1913.

ROBERT SPENCER THOMAS, M. Am. Soc. C. E.[1]

DIED MAY 11, 1941

Robert Spencer Thomas, the son of Spencer Farrington Thomas and Kate Pugh (Fanning) Thomas, was born in Brownsville, Tenn., on December 27, 1882.

After attending the University of Tennessee in Knoxville, Tenn., in 1900–1901, "Rob" Thomas was appointed to the United States Military Academy (West Point, N. Y.) from the State of Tennessee on July 29, 1901. He was graduated number nine in a class of one hundred and fourteen, and was commissioned Second Lieutenant, Corps of Engineers, United States Army, on June 13, 1905. By his classmates and friends he was affectionately known as "Tommy."

Immediately following graduation leave, June 13, 1905, to September 30, 1905, Lieutenant Thomas was assigned to the First Battalion of Engineers and as assistant engineer, Moro Province, Philippine Islands, remaining there until August, 1907, when he returned to the United States. After a short tour at Fort Mason, California, he was assigned to duty with the Second Engineer Battalion and as a Student of The Engineer School, Washington Barracks, D. C.

He was promoted to First Lieutenant, Corps of Engineers, on February 28, 1908, graduated from The Engineer School in May, 1909, and in June, 1909, he was assigned to the Third Engineer Battalion, Fort Leavenworth, Kansas, where he remained until October, 1911, when his station was changed to Fort Riley, Kansas. There he attended The Mounted Service School, from which he was the first Engineer Officer to graduate. He had always been an ardent horseman and during this time his enthusiasm for polo was given a lasting impulse. While a student officer at this school he acted as instructor in "Military Explosives and Demolitions."

On July 8, 1912, Lieutenant Thomas reported for duty as assistant District Engineer, Kansas City, Mo., and on February 27, 1913, he was promoted to Captain, Corps of Engineers. While on duty at Kansas City, he was assigned to additional duty as Inspector-Instructor, Organized Militia of Colorado. He was relieved of assignment and duty at Kansas City on September 19, 1914. On September 25 he reported for duty at New York, N. Y., as Inspector-Instructor of Organized Militia and was relieved of this duty in August, 1916. While on duty in New York City, he was married to Eleanor Marshall on December 28, 1915. Captain Thomas' soul was full of the milk of human kindness and he was liked and admired by the civilian soldiers with whom he worked with so much patience and enthusiasm.

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[1] Memoir prepared, under the direction of C. L. Adcock, Lieutenant-Colonel, Corps of Engineers, by the personnel section of the Office of the Chief of Engineers, U. S. Army, with the active cooperation of Maj. Francis J. Wilson, M. Am. Soc. C. E., and Col. Ernest Graves.

On August 19, 1916, Captain Thomas reported at El Paso, Tex., as assistant to the District Engineer, El Paso District, and later he was assigned to The Engineer Depot, Eagle Pass, Tex., where, after a few days, he was assigned to duty as instructor, Officers' Training Camp at Fort Logan H. Roots, Arkansas, on April 30, 1917. He was relieved from this duty on August 24, 1917.

Captain Thomas was promoted to Major, Corps of Engineers, on May 15, 1917, and was promoted to Lieutenant Colonel, Corps of Engineers, National Army, August 5, 1917. On August 25, 1917, he reported at Camp Pike, Arkansas, for duty with the 312th Engineers, where he remained until March 31, 1918. He was promoted to Colonel, Corps of Engineers, National Army, on April 9, 1918.

Colonel Thomas was Commanding Officer of the 22d Engineers (Light Railway Construction) at Camp Sheridan, Alabama, from April to June, 1918, and Commanding Officer, 213th Engineers at Fort Benjamin Harrison, Indiana, and at Camp Forrest, Georgia, from April, 1918, to August, 1918. He then attended a Special Staff Course at The Army War College, Washington, D. C., and was awarded a Certificate of Qualification as "Division Engineer" on September 30, 1918, after which he returned to Camp Forrest, Georgia, and rejoined his regiment.

Colonel Thomas proceeded with the 213th Engineers to their new station at Camp Lewis, Washington, where he remained as Commanding Officer of that organization until February 24, 1919. It was always a source of regret to Colonel Thomas that he did not go overseas in the World War because he was an ardent soldier. However his talents were most useful in training troops in the United States.

In March, 1919, Colonel Thomas reported for duty as District Engineer, Mobile, Ala., and remained on that duty until August, 1920. On July 14, 1919, he was discharged as Colonel, only, and reverted to Major, Corps of Engineers.

On August 28, 1920, Major Thomas reported for duty at Fort Leavenworth, Kansas, where he remained until June 28, 1922. While stationed at Fort Leavenworth he attended the School of the Line, from which he was graduated in June, 1921, and The General Staff School, from which he was graduated in June, 1922. He was particularly interested in perfecting himself for the military profession which he had chosen when he went to West Point.

From July, 1922, to July, 1924, Major Thomas was on duty at Fort Humphreys, Virginia, as instructor in The Engineer School, and director of the Department of Military Art. From July, 1924, to June, 1928, he was detailed with the Militia Bureau, Washington, D. C., after which he was assigned to duty as District Engineer, Buffalo, N. Y.

Major Thomas was promoted to Lieutenant Colonel, Corps of Engineers, on March 2, 1929. From June, 1929, to October, 1930, he was Department

Engineer, Philippine Department, and District Engineer, Manila, Philippine Islands, when due to a tropical infection he was compelled to return to the United States. On December 4, 1930, he reported for duty at San Francisco, Calif., where he served as assistant to the Division Engineer, South Pacific Division, until March 1, 1931, when he was assigned to duty as District Engineer, San Francisco. He was designated a Member of the California Debris Commission from March 21, 1931, to July 13, 1931, and a Member of the San Francisco Harbor Line Board for the same period. In August, 1931, he reported for duty as a Student, The Army War College, Washington, D. C., and was graduated on June 27, 1932. It had always been one of his cherished ambitions to be a graduate of the War College. From June 30, 1932, to December 5, 1935, he was District Engineer, in Mobile, and while serving in this capacity he was promoted to Colonel, Corps of Engineers, on August 1, 1935. During his tour the major part of the Gulf and Intracoastal Waterway in the Mobile District was planned and constructed.

On January 28, 1936, Colonel Thomas reported for duty as Commanding Officer, Third Engineer Regiment, and Department Engineer, Hawaiian Department, where he was able in this dual capacity to promote and construct with the Third Engineers an essential military road over Kole Kole Pass.

On July 21, 1938, Colonel Thomas reported for duty as District Engineer, Washington, D. C., where he remained until December 18, 1940. While on duty in Washington, D. C., he was appointed a representative on the Engineering Commission for the Washington National Airport at Gravelly Point. As such he planned and supervised the intricate engineering construction of the base fill for the world's finest airport. He was also in charge of reclamation of Anacostia River Flats and the Washington Aqueduct, Washington Aqueduct filtration plants and appurtenances. While on this last civilian assignment "Tommy" endeared himself to the people in and around Washington where he left a host of friends.

Colonel Thomas' last duty was as he would wish it to be—a military assignment as Engineer, Second Army at Memphis, Tenn., where he reported for duty on December 22, 1940. However, within less than a year he succumbed with his last and fatal illness. He is survived by his widow, Mrs. Eleanor Marshall Thomas.

Colonel Thomas was an Engineer Officer of superior intelligence, who possessed unusual technical knowledge, and fine qualities of leadership and cooperation. His long and faithful service in the United States Army was an example of duty well performed. He never failed in his loyalty to his superiors and to his country. The Corps of Engineers laments the passing of a lovable, able, and valued officer. Colonel Thomas was buried in Arlington National Cemetery, in Virginia, with full military honors befitting his rank.

Colonel Thomas was elected a Member of the American Society of Civil Engineers on July 6, 1920.

GEORGE WILLIAM TILLSON, M. Am. Soc. C. E.[1]

DIED MAY 13, 1940

With the passing of George William Tillson a host of people who counted it a privilege to know him lost a true and loyal friend. His close friends, particularly, will miss his devoted friendship. The engineering profession also lost a celebrated member, one who was always fair-minded, just, strict, meticulous, with a keen sense of humor, and a mind that was a vast storehouse of statistics and engineering anecdotes. He was always a champion of the engineering profession and deemed it a privilege to be an engineer. He was ever ready to give a helping hand to a brother engineer, or kindly, encouraging advice to any of the younger generation studying the subject.

George William Tillson was born in Thomaston, Me., on December 18, 1852, the son of Perez Tillson and Harriet (Collins) Tillson, and was a direct descendant (eighth generation) of Edmund Tillson, known to have been in Plymouth, Mass., in 1638.

Young George Tillson was brought up in the Congregational Church, and his early life was typical of the hardy New England boy with the chores, farm work, hunting, and life in the outdoors that gave him the health and vigor that he retained until he was in his early eighties.

He received his grammar and high school education in Thomaston and after graduation from high school taught in nearby country schools. He entered Bowdoin College, at Brunswick, Me., in 1873 and was graduated in 1877. He was a brilliant and outstanding scholar during his college course, and was a member of the Zeta Psi Fraternity. In his senior year he was elected to Phi Beta Kappa. After his graduation from Bowdoin, he taught Latin, Greek, and mathematics at Nicholas Academy in Dudley, Mass. Then, in 1879, he taught school at Rumford, Me. In 1880 he received the degree of Civil Engineer from Bowdoin College.

In 1880 he accepted an engagement as assistant engineer on sewer construction work in Memphis, Tenn. When this work was completed he went to Kalamazoo, Mich., on the same class of work and while working there was offered the position of assistant engineer in charge of sewer and paving work for the City of Omaha, Nebr., which he accepted in 1881. In 1887 he was made city engineer, which position he held for five years.

While working as assistant engineer in Omaha, he met Mary Elizabeth Abbott, of Lancaster, N. H. After he was made city engineer they were married, the ceremony taking place at Lancaster on October 5, 1887. They had one daughter, Madalene Abbott Tillson (Mrs. N. R. Clark), who survives him.

For three years after he terminated his work in Omaha, Mr. Tillson engaged in private consultation, engineering, and contracting work—from 1892 to 1895. Then he accepted a position as assistant engineer in charge of pavement construction for the Department of City Works in Brooklyn, N. Y. He held this position until 1902, when he was appointed chief engineer of the Bureau of Highways for the Borough of Brooklyn. In June, 1907, he was transferred to the same position for the Borough of Manhattan, and in 1911 was transferred to the position of consulting engineer to the Borough President of Brooklyn, which position he held until June, 1918.

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[1] Memoir prepared by Ralph H. Gage, M. Am. Soc. C. E.

In December, 1918, he was appointed by the Society as one of a commission of engineers to visit France and confer with representatives of the French Congress who were considering different projects to advance France as a commercial nation after the war.

During his lifetime Mr. Tillson was associated in the following capacities with technical and scientific societies: President of Nebraska Association of Engineers; member of Brooklyn Engineers Club; member of Municipal Engineers of the City of New York; past-president and secretary of American Society of Municipal Improvements; director and past-president of American Road Builders Association; member of Western Society of Engineers; and member of Permanent International Association of Road Congresses.

He was the author of "Street Pavements and Paving Material" (first edition, 1900; second edition, 1912). This book, the first of its kind, became well known and was extensively used as a reference work, and also as a text-book in several universities. He also wrote four chapters in the "American Highway Engineers' Handbook" of 1919, and contributed many technical papers to engineering societies and magazines.

Mr. Tillson retired from active business in 1918. In 1919 Bowdoin College honored him with the degree of Doctor of Science.

Mr. Tillson spent his remaining years at La Grange, Ill., happily surrounded by his many friends, his grandchildren, and his son-in-law and daughter, Mr. and Mrs. N. R. Clark, with whom he lived. He enjoyed his family and their friends, the social life of the village, golf at the country club, and made a place for himself wherever he went. His two grandchildren were always a great joy and interest to him, and he was always a source of knowledge and inspiration to them. He took an active part in village affairs, serving as secretary of the La Grange Village Plan Commission, and also on the Board of Directors of the La Grange Public Library for twelve years. He served as secretary-treasurer for eight of these years and as president for the four years previous to his death.

One may obtain a fairly accurate insight into his character and philosophy of life from an extract from his autobiography, which reads as follows:

> "In a long life everyone must have made mistakes, must have made decisions that he would have changed if he had known what was coming afterwards; but if he had studied his problem carefully and honestly, he should not be unhappy if the decision did not always prove to be correct. No man is infallible. He may be sorry for his errors, but they should not make him unhappy if he always acted according to his best judgment. This may seem to be a too optimistic philosophy, but if lived up to, people

would be much happier than the majority of them are today. I do not mean that I have always accomplished this, but the nearer that I have approached it, the happier I have been. If I can continue in this way until my end comes, I feel that I should be content."

Mr. Tillson was elected a Member of the American Society of Civil Engineers on October 7, 1896, and served as Director (1907–1909) and Treasurer (1917–1918).

GUSTAV ADOLPH TRETTER, M. Am. Soc. C. E.[1]

DIED MARCH 19, 1940

Gustav Adolph Tretter was born in Neuhäusel (later Nehäus), Germany, on July 27, 1859. He attended the Royal Bavarian Gewerbe School, and was graduated from the Construction Department of the Technical Institute at Kaiserslautern, Germany, in 1879. From 1880 to 1882 he studied civil engineering at Polytechnicum, in Munich (later München), Bavaria.

In 1883 Mr. Tretter came to the United States and shortly thereafter became an American citizen. He obtained employment with the Phoenix Bridge Company of Phoenixville, Pa., where he was immediately given the opportunity of entering the field of construction for which his schooling had prepared him. His unusual energy, his capacity for detail, and his intense interest in his chosen profession brought rapid advancement from field clerk to assistant foreman and to foreman in charge of erection of railway bridges and other steel structures.

In 1889 Mr. Tretter was transferred from the field to the office, and until 1894 he was engaged in making designs of derricks, travelers, other types of erection equipment, and construction layouts for the Erection Department of the Phoenix Bridge Company. From 1894 to 1895 he was superintendent of erection for the Lake Street Elevated Extension, in Chicago, Ill., from the river to Wabash Avenue.

In November, 1895, he was made assistant superintendent and erection engineer of the Phoenix Bridge Company. In this position he had full responsibility for the planning of all erection, and the designing of equipment required for the erection of all structures fabricated and erected by the company. He was frequently called upon to direct work in the field.

On January 1, 1908, Mr. Tretter left the Phoenix Bridge Company to accept the position of erection manager with the Virginia Bridge and Iron Company, in Roanoke, Va., and continued in this capacity until July 1, 1922, when he was made vice-president in charge of engineering and erection. During this period, he planned and supervised the erection of numerous steel structures throughout the South and Southwest.

Mr. Tretter retired from active service on August 1, 1929, and continued to make his home in Roanoke until he died on March 19, 1940. He was a member of the Shenandoah Club of Roanoke, the Roanoke Country Club, and the Rotary Club. Known as "Gus" to his intimate friends, he was a popular member of these organizations and was always interested in the civic affairs of the community.

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[1] Memoir prepared by C. W. Ogden, M. Am. Soc. C. E.

He was conscientious and sincere in all of his relations with his associates and employees, and his sympathetic understanding of their problems endeared him to all and inspired their complete loyalty and cooperation. He was never too busy to take an active interest in their welfare. These qualities made him a man of unusual ability as a leader, and contributed much to his success as an engineer. He was highly respected by the members of his profession throughout the United States. Mr. Tretter enjoyed a long life of engineering activity, followed by a few well-earned years of retirement.

He was married to Gertrude S. Ewing, of Phoenixville, who survives him.

Mr. Tretter was elected a Member of the American Society of Civil Engineers on April 6, 1904.

RUDOLPH PETER JOHANN TUTEIN-NOLTHENIUS, M. Am. Soc. C. E.[1]

DIED NOVEMBER 27, 1939

Rudolph Peter Johann Tutein-Nolthenius was born on June 6, 1851, in Abcoude, Netherlands. In 1872 he received the degree of Civil Engineer from the University of Delft, at that time called The Polytechnical School.

After graduation he was employed for some time on the construction of the national railway system. In 1874, having passed the necessary examination with honor, he was appointed assistant engineer of the corps of "Waterstaat," Department of Civil Engineering and Building. In this governmental service he held several different positions and assisted in, or supervised, the construction of important engineering works, among which were the following: (1) In 1882, with an officer of the Dutch Navy, he made a survey of the ocean currents along the coast for hydraulic engineering purposes; (2) in 1883 he planned and directed the cutting of a new outlet for the Maas (Meuse) River; and (3) in 1895 he was appointed supervisor of the northern part of the province of Gelderland.

In 1902, meanwhile having attained the rank of chief engineer, first class, Mr. Tutein-Nolthenius resigned from the Department of "Waterstaat" to become director of the banking firm of Tutein-Nolthenius and De Haan, in Amsterdam, Netherlands, which position he held until 1909.

Although he no longer held an engineering position, he kept up his relations with the Netherlands engineering world. As a member of the "Koninklijk Instituut van Ingenieurs" (Royal Institution of Netherlands Engineers), he was a faithful visitor to the meetings, where he appeared many times as a speaker with a great sense of humor. *De Ingenieur* (*The Engineer*), the weekly organ of the Institute, which Mr. Tutein-Nolthenius as secretary of the Committee of Preparation helped to publish for the first time in 1886, held his lasting interest. From 1886 to 1939 no volume appeared without a valuable contribution from his pen.

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[1] Memoir prepared by Wouter Cool, General Secretary, Koninklijk Instituut van Ingenieurs, The Hague, Netherlands.

Mr. Tutein-Nolthenius was a versatile writer; in addition to technical subjects, he wrote books, brochures, and articles on social, economic, and historical subjects, as well as purely literary subjects. From 1897 to 1919 he was one of the editors of the famous literary magazine, *De Gids* (*The Guide*), in which many of his best and most copious studies appeared.

He loved to travel and visited the United States in 1899. He returned from this trip full of enthusiasm for what he had seen and experienced, and for the warm hospitality shown him by his American colleagues, who repeatedly remarked on his strong likeness to the late Theodore Roosevelt. At a conference of the Royal Institution of Engineers in 1900, he gave his impressions of that journey.[2] He expressed his great admiration for the engineering works he saw there and for the method of work used in factories. He was no less enthusiastic over the technical instruction in the United States, in which practical training appeared to play such an important rôle. In later writings, and in meetings of diverse committees on education, he repeatedly pleaded for instruction along the American lines, although he realized that not all of the methods could be applied in the Netherlands.

He never forgot that journey of 1899. Twenty years later he wrote in *De Ingenieur,* in an article entitled "Americana," that he still had an American aphorism hanging above his desk which read: "If what you did yesterday appears important to you today, then you have not accomplished very much today." It may be noted that he wrote biographical sketches of two great Americans for the Netherlands magazine, *Mannen van Beteekenis* (Men of Significance): Andrew Carnegie (1902) and Theodore Roosevelt (1904).

He was extremely appreciative of his election to membership in the Society through the introduction of the late Edward P. North,[3] M. Am. Soc. C. E., whom he had met at International Navigation Congresses.

Although Mr. Tutein-Nolthenius lived to a ripe old age, he retained a great vitality and clarity of mind. Since 1919, he had made his home in Switzerland, but he still had a warm interest in everything which took place in his fatherland, as is witnessed by his wide correspondence with old friends. To his sorrow, these passed away one by one, but there remained numerous younger colleagues who were proud to be the recipients of his worth-while, beautifully styled letters written in his powerful handwriting.

After the death of his first wife, Alieda Maria Tutein-Nolthenius (his cousin), in 1910, Mr. Tutein-Nolthenius was married in 1920 to Lilas Goergens, of Switzerland, who survives him. Thereafter he made his home at La Tour de Peilz (Vaud), where he died after a short illness on November 27, 1939.

Mr. Tutein-Nolthenius was elected a Member of the American Society of Civil Engineers on December 6, 1899.

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[2] "Nieuwe Wereld" (New World), by R. P. J. Tutein-Nolthenius, 1900 (2d Ed., 1902).

[3] For memoir, see *Transactions*, Am. Soc. C. E., Vol. LXXV, December, 1912, p. 1167.

SAMUEL MATTHEWS VAUCLAIN, M. Am. Soc. C. E.[1]

DIED FEBRUARY 4, 1940

Samuel Matthews Vauclain, son of Andrew C. and Mary A. (Campbell) Vauclain, was born on May 18, 1856, in the town of Port Richmond, which has since been incorporated within the limits of the City of Philadelphia, Pa.

Shortly after his birth the family moved to Altoona, Pa., where his father was employed in the Pennsylvania Railroad Shops. At that time Altoona consisted of perhaps a dozen dwellings.

Mr. Vauclain was educated in the public school at Altoona, but was not graduated, although his teachers urged him to go to college. Full of health and vitality, he was always fond of manual work and therefore started to work in the shops of the Pennsylvania Railroad Company when only sixteen years of age. At the age of seventeen he was entered as an indentured apprentice to the machinist trade. Upon completion of the four years apprenticeship, he was made assistant gang foreman in the locomotive frame shop.

At an early age—even before becoming an apprentice—he decided upon a course of action that greatly influenced his career. He resolved that whatever advancement he would make in life would not be made through influence; for that reason he never joined any social organization, lodge, or church.

In 1882 the Pennsylvania Railroad Company sent him to inspect some locomotives which were being built for them by the Baldwin Locomotive Works. Upon completion of this assignment he entered the employ of the Baldwin Locomotive Works. Advancement was rapid, and, at the age of thirty, he was promoted to mechanical superintendent of the entire plant.

On January 1, 1896, Mr. Vauclain became a member of the firm; on July 1, 1911, he was made vice-president, in recognition of his efforts to promote the growth and products of the works; on May 19, 1919, he was elected president, and on March 28, 1929, was elected chairman of the Board of Directors of the Baldwin Locomotive Works, which position he retained until his death.

From early childhood he always had a great interest in locomotive construction, fostered, no doubt, by his father, who helped to build "Old Ironsides," the first locomotive built by the late M. W. Baldwin, and one of the first ever built in America.

It was during the period of the first World War that Mr. Vauclain showed his greatest ability as an executive and organizer. In addition to supplying a lot of 5,551 locomotives for all the belligerent nations, including the United States, he also generally directed the erection of plants for the manufacture of army rifles and ammunition. These plants produced two thirds of the rifles used by the American Expeditionary Forces in combat and almost 7,000,000 shells, at the rate of 6,000 rifles and 15,000 shells per 24-hr day. A number of 14-in. railway and 7-in. caterpillar gun mounts were also constructed. The aggregate value of the war contract approximated $250,000,000.

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[1] Memoir prepared by Charles C. Campbell, Assoc. M. Am. Soc. C. E.

Mr. Vauclain served as chairman of the Locomotive and Car Committee on the Council of National Defense, and also as chairman of the Special Advisory Committee on Plants and Munitions of the War Board during the period of the World War.

He was a born leader, a dominant character, and a hard worker. A number of addresses made before various railway, engineering, and civic societies, together with interviews with representatives of many popular publications, have been compiled into a book, published in 1924, and entitled *Optimism*.

Although often referred to as a statistician, politician, economist, banker, broker, and orator, Mr. Vauclain modestly preferred to be known merely as a locomotive builder. The honorary degree of Doctor of Science was conferred on him by the University of Pennsylvania, at Philadelphia, Pa., and Worcester Polytechnic Institute, at Worcester, Mass.; and the honorary degree of Doctor of Laws by Villanova College, at Villanova, Pa. He had the distinction of being the only person to receive the John Scott Medal award more than once —in 1891 and again in 1931. Mr. Vauclain was the recipient of decorations from the United States, French, Italian, and Polish governments.

He was a director of several banks and financial and industrial organizations; manager of many hospitals and welfare organizations; and a member of numerous civic, technical, and educational societies.

On April 17, 1879, Mr. Vauclain was married to Annie Kearney. Of the three sons and three daughters born of the union, he is survived by two sons and two daughters, together with fifteen grandchildren and four great-grandchildren.

Mr. Vauclain was elected a Member of the American Society of Civil Engineers on December 5, 1911.

CHARLES EDWIN WELLS, M. Am. Soc. C. E.[1]

DIED AUGUST 4, 1940

Of early New England ancestry, Charles Edwin Wells was born in North Adams, Mass., on April 27, 1858, the son of Daniel M. Wells and Mary M. (Sly) Wells. His family had been identified with that section of the state for 200 years. After completing his course in the local public schools, he was graduated from the Drury Academy, in North Adams, in 1876 and in the fall entered Worcester Polytechnic Institute, in Worcester, Mass., from which he received the degree of Bachelor of Science in 1880.

Upon his graduation he became assistant engineer and later principal assistant engineer with the old Troy and Greenfield Railroad and Hoosac Tunnel (later a part of the Boston and Maine Railroad System). This line, originally 45 miles long and state-owned and operated, passed through North Adams. He was placed in charge of double-tracking the line through the tunnel and the approaches in each direction.

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[1] Memoir prepared by Arthur W. Tidd, M. Am. Soc. C. E.

From March, 1886, to February, 1887, he served with the Chicago, Burlington and Northern Railroad Company, with headquarters at various times in Oregon, Wisconsin, and Minnesota. During the latter part of this employment he held the position of superintendent of bridges.

He left this company to become division engineer on the Chicago, Santa Fé, and California Railroad in charge of construction of 60 miles of line through Galesburg, Ill. He held the position until July, 1888. The following year he was locating engineer on a part of the line of the Sault Ste. Marie and Southwestern Railroad in Wisconsin, and until early in 1891 he was on maintenance and location with the Chicago and North Western Railway in Wisconsin.

From April, 1891, to October, 1893, he was engineer and superintendent of construction for MacArthur Brothers Construction Company of Chicago, Ill., on large contracts with the Chicago and Eastern Illinois Railroad Company; Grand Trunk Railway System in Canada; Toledo, Walhonding Valley and Ohio Railroad Company; and Ohio Southern Railroad Company. The year 1893 saw the general completion of railroad construction in the West, and many engineers were forced into other lines of engineering work. In 1894 Mr. Wells was engaged in private practice in Davenport, Iowa, and from April to August, 1895, he was superintendent of waterworks in Galesburg.

On August 1, 1895, he accepted the position of division engineer with the Metropolitan Water Board (later Metropolitan Water and Sewerage Board) of Boston, Mass., and moved to Clinton, Mass., where construction was about to be undertaken for the large Wachusett Reservoir to augment the water supply for Boston and other cities in the metropolitan district. He was placed in charge of the third division of the aqueduct comprising about 3 miles of covered aqueduct, 10.5 ft by 11.5 ft, and about 3 miles of masonry-lined open channel.

On April 21, 1899, he was transferred to the Reservoir Department and placed in charge of the removal of about 6,900,000 cu yd of topsoil from the reservoir site, an area of about 7 sq miles. Beginning on September 21, 1903, all the work in the Reservoir Department was placed under his direction as department engineer. He held this position until the close of 1904, when he resigned so that he might accept an appointment with the Reclamation Service, U. S. Department of the Interior.

Mr. Wells entered the Reclamation Service on January 4, 1905, having been appointed from a competitive civil service examination list for supervising engineer. He was among the first three on the list. He was placed in charge of the construction of irrigation works in southern Wyoming, Nebraska, and South Dakota, principal among which was the North Platte project (North Platte River), including the Pathfinder Dam and Reservoir. This is one of the largest irrigation projects and serves 425,000 acres of irrigable land extending for a distance of approximately 100 miles. The Pathfinder Dam, a masonry structure 218 ft high, is located about 50 miles southwest of Casper, Wyo. The long Interstate Canal was a part of the project. The Belle

Fourche project in South Dakota, with its dam, reservoir, and canal systems, lay within his territory, as did also the Gunnison Tunnel in Colorado. It was during the three years of his work in that section that he witnessed the passing of the frontier with its saloons, open gambling, and cattle rustling, an era when only the man with the quickest "draw" lived to enjoy his old age. With the introduction of modern irrigation had come a most progressive advancement, and many communities sprang up that are now highly developed. Mr. Wells, always a keen observer of human nature in all its relationships, never tired of relating incidents and experiences that came to him during that period.

He resigned from the Reclamation Service at the close of 1907 to accept an appointment as division engineer with the Board of Water Supply of the city of New York, on its new Catskill water supply, then in its early stages. This brought him back again to the East for what was to prove his greatest engineering accomplishment. He began work with the Board on December 6, 1907; was assigned to the Hill View Division; and spent the ensuing nine years beginning and completing contracts aggregating more than five and one-half million dollars. The division, 4.3 miles in length, was located in Yonkers, N. Y., and included principally 1.1 miles of steel pipe siphon 11 ft 3 in. in diameter; 2.3 miles of 16-ft 7-in. pressure tunnel aqueduct; and the Hill View Reservoir, an artificial, concrete-lined reservoir, roughly rectangular, 3,000 ft long, 1,500 ft wide, and 36.5 ft deep, with rolled earth embankments. The reservoir began filling with Catskill mountain water on November 30, 1915, but it was another year before the work on the division was completed. From November, 1916, to July, 1917, he was engaged on special work connected with water power claims along the Esopus and Schoharie creeks.

During World War I, Mr. Wells' long experience on large construction projects in various parts of the United States was utilized by the federal government. On July 31, 1917, he resigned from the Board of Water Supply to become resident engineer on the construction of Camp Merritt, New Jersey, an embarkation camp near Englewood to accommodate 30,000 men. Work began on August 15, 1917, and the camp was fully occupied on December 25, 1917, complete with water and sewerage systems, roads, and a sewage disposal works. In spite of the tremendous amount of detail involved, Mr. Wells found time to study the layout of roads and buildings and to direct the contractor assigned to the task of clearing the site so that only the trees that actually fell within the limits of those features were cut down, with the result that many well-deserved compliments were later paid Camp Merritt for its pleasing appearance and welcome shade.

In 1918, at the age of 60, he was successful in another competitive examination and received an appointment as supervising plant engineer for the U. S. Shipping Board. He was assigned to San Diego, Calif., where a government-owned shipyard plant for the construction of reinforced-concrete ships was being built. He held that position until late in 1919 when he retired to his old home in North Adams.

Shortly thereafter he opened a private engineering office, specializing in land court work and the relocation of old property lines. Again passing a civil service examination in 1924 with a high rating, he was appointed and served the City of North Adams in the capacity of city engineer until the position was abolished in 1925.

From that time Mr. Wells' interest centered more and more around his home affairs and his church. A Congregationalist, he had been active in church matters in every community in which he had been a resident, as well as in civic matters generally. He was secretary of the Class of 1880 of Worcester Polytechnic Institute for many years and kept in touch with all of its members until his death, sending each one a Christmas greeting every year.

A man of inflexible integrity, complete loyalty, respected by all with whom he came in contact, skilled in the handling of men, and possessed of a profound knowledge of human nature, Charles E. Wells was always ready and willing to help wherever help was needed. One of his subordinates of some years ago writes

> "With a fairness characteristic of him in dealing with the contractor on important and difficult controversies, and with the wisdom native and developed through his long experience in handling construction, he taught us younger men many a valuable lesson which has been helpful in succeeding years";

and one of the ablest engineers of recent times writes:

> "He has unusual executive ability. In carrying out the extensive work under my direction, I always felt the greatest confidence, because of his ability to get the work done and get it done well without occasioning friction."

Uncompromising in his stand for honesty, Mr. Wells was charitable in his judgment of the frailties of his fellow men. Standing squarely on his own, thoroughly experienced in his judgment of construction materials and in construction methods, he asked no favors from any one and met the contractors on their own ground. His fairness, firmness, and quiet sense of humor always stood by him and brought him triumphantly through many difficult situations. Never given to the use of profanity, he was quite competent to express himself clearly when the occasion demanded. His long and distinguished career in the engineering profession, covering some of the nation's most important construction projects, attracted the attention of the editors of *Who's Who in America,* and an account of his life appeared in that publication for 1934–1935.

He often remarked to his family and friends that it was the romance of engineering that appealed to him, and he was fond of describing the thrill that he experienced as he went into new territory, virgin forests, or barren prairies on a project that was going to help make the world a better place in which to live. His pleasure came from watching the project grow, develop, and finally come to full fruition—romance rather than mathematics, although he carried out his work with exactness and would brook nothing in the way of carelessness or slipshod methods. He demanded the same thoroughness and precision of himself that he expected of his subordinates.

Mr. Wells was married to Katherine Belden of Galesburg, on February 19, 1891. Mrs. Wells survives him, together with three daughters, Mrs. Leon R. Whipple (neé Katherine Zeruah Wells), Sarah Frances Wells, and Margaret Adams Wells. A son, Marcus Belden Wells, died in 1934. He is also survived by an own sister, Sarah A. Wells, by a half-sister, Mary Grace Wells, and two half-brothers, Daniel M. Wells, Jr., and Harry O. Wells.

He was a life member of the Society, a member of the Boston Society of Civil Engineers, and a former member of the Municipal Engineers of the City of New York.

Mr. Wells was elected a Member of the American Society of Civil Engineers on June 1, 1892.

GEORGE FURBUSH WHITTEMORE, M. Am. Soc. C. E.[1]

DIED JULY 15, 1941

George Furbush Whittemore was born in Wollaston, Mass., on July 30, 1876, the son of Luther Whittemore. His mother was an Adams.

He attended the Adams Academy in Quincy, Mass., from 1888 to 1892. From 1892 to 1894 and from 1904 to 1905 his education was continued at the Massachusetts Institute of Technology, at Boston, Mass., and at Armour Institute of Technology (later Illinois Institute of Technology), at Chicago, Ill., respectively.

From 1896 to 1897 Mr. Whittemore was draftsman and assistant engineer on contract work for maintenance of way and structures for the Mexican Central Railroad, Buena Vista, Mexico, D. F., and from 1897 to 1898 he was assistant engineer and principal assistant engineer on sewerage construction and water works design for the state engineer, Jalisco, Guadalajara, Mexico. He continued work in Mexico from 1898 to 1901, in charge of lines and grades for breakwaters, dredging, sewerage works, and a pumping plant in Vera Cruz, for S. Pearson and Son, London, England. While he was employed by S. Pearson and Son, he was also subdivision engineer in charge of rehabilitation of the Tehuantepec National Railway from Puerto, Mexico. Mr. Whittemore returned to the United States in 1901 and became assistant engineer for the Massachusetts Highway Commission, in Boston.

His first work with the United States Engineer Department was in Savannah, Ga., from 1902 to 1904, where he served as surveyor and junior civil engineer on river and harbor work, including surveys of ocean bars, Savannah Harbor, Darien River, and Brunswick Harbor. He was in local charge of dredging.

For a short period from 1904 to 1905 he was assistant engineer on right of way, reconnaissance, and maintenance of way for the Central of Georgia Railway Company.

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[1] Memoir prepared by River and Harbor Subdivision, U. S. Engr. Office, Los Angeles, Calif.

He returned to service with the U. S. Engineer Department, First District, New York, N. Y., in 1905, as junior civil engineer on harbor line surveys and examination and report on Harlem Kills. He was in charge of dredging surveys of the Coney Island Channel. From 1906 to 1910 he was junior engineer for the Engineer Department in Portland, Ore., in local charge of south jetty construction at Fort Stevens, Ore. He was transferred in 1910 to the Engineer Department in Hawaii, where he served until 1914 as junior civil engineer in local charge of location and construction of United States railway lines to Fort Armstrong and the United States quarry, Oahu; dredging of Kahului Harbor; and construction of the breakwater at Hilo Harbor.

He was again engaged in dredging operations for the Engineer Department, Little Rock District, Arkansas, from 1914 to 1915, when he was in field charge of dredging the 6-ft channel of the Arkansas River below Pine Bluff. He continued service in the Engineer Department in San Francisco, Calif., from 1915 to 1918 as assistant engineer in local charge of the suboffice in Eureka, Calif., on jetties, breakwater, and surveys north of Cape Mendocino to the Oregon line, including the Eel, Mad, Klamath, and Smith rivers, in California.

Mr. Whittemore was transferred to the United States Bureau of Reclamation, at Yuma, Ariz., and from 1918 to 1919 was superintendent of construction and operations. The principal work was levee revetment, on the Lower Gila and Colorado rivers on the Arizona side, including operation of the Yuma quarry.

In 1919 Mr. Whittemore returned to the Engineer Department as assistant engineer in Mobile, Ala., where he was in immediate charge of surveys and report for the lock and dam extension, Upper Warrior River and Mulberry and Sipsey Forks, Ala. He returned to the San Francisco office of the Engineer Department as assistant engineer in 1920 and shortly thereafter was promoted to principal assistant engineer for the district and division offices on river and harbor work, including preliminary reports. In 1928 he was transferred to the St. Louis, Mo., office of the Engineer Department and promoted to the grade of engineer (civil), in immediate charge of reports for the district. From 1930 to 1932 he served as senior engineer in the South Pacific Division office, Engineer Department, San Francisco, on the navigation and economic features of the Salt Water Barrier, and consultant on initial concrete repair work on the Humboldt Jetties and the Monterey breakwater. From 1932 until his death Mr. Whittemore was engaged in a supervisory capacity as senior engineer on harbor and flood-control reports, breakwater, and dredging.

His service was a valuable contribution to the Engineer Department, particularly in connection with the construction of the jetties at the mouth of the Columbia River, in the states of Washington and Oregon, and the 12,500-ft detached breakwater in the Los Angeles-Long Beach (Calif.) harbors.

Mr. Whittemore was a member of the Masonic fraternity, Wilshire Lodge No. 445; Al Malaikạ Temple of the Shriners; Scottish Rite of Freemasonry; Permanent International Association of Navigation Congresses; Society of

American Military Engineers; and National Federation of Federal Employees. He was a registered civil engineer in the state of California.

Mr. Whittemore served as Private in the Massachusetts Cavalry, and Ensign and Lieutenant (J.G.) in the Naval Reserves, Georgia.

He died on July 15, 1941, at the Queen of Angels Hospital in Los Angeles after approximately five weeks' illness. He is survived by his widow, Henrietta Whittemore; three children, Ben Henderson, Wright Henderson, and Bernice Henderson (Mrs. Emmett Flood); a grandson and granddaughter; one brother, Lawrence Whittemore; and three sisters, Mrs. Lemuel Hodgkins, Mrs. Clinton Willey, and Mrs. Raymond Van Tassel.

Mr. Whittemore was elected an Associate Member of the American Society of Civil Engineers on January 17, 1916, and a Member on October 15, 1923.

MARSHALL WILLIAMS, M. Am. Soc. C. E.[1]

DIED DECEMBER 5, 1940

Marshall Williams was born in Blackwood, Camden County, N. J., on July 24, 1873, the son of Benjamin and Abigail (Turner) Williams. He was graduated from Rutgers University, in New Brunswick, N. J., with the degree of Bachelor of Science in 1894, and entered the employ of Haupt and Franklin, in Philadelphia, Pa., where his work consisted of surveying, municipal engineering, and inspection of buildings during 1895 and 1896.

From 1896 to 1898 he was assistant engineer for the Pittsburgh Testing Laboratories, inspecting mill material and finished structures. During the period 1899–1900, he was chief shop inspector for the Pencoyd Iron Works, in Pencoyd, Pa. (A. and P. Roberts Company). In January, 1901, following the merging of the Pencoyd Iron Works with the American Bridge Company in May, 1900, Mr. Williams was appointed assistant operating manager of the Eastern Division of the American Bridge Company. In June, 1901, he was appointed to the President's staff as engineer assistant to the President with duties as investigator of general shop and office practice throughout the company. From February, 1902, to April, 1904, he was operating manager of the Pittsburgh (Pa.) Division, in charge of the prosecution of work in nine shops. He was given a special engineering assignment from April, 1904, to October, 1905, that necessitated his relinquishing his management duties for the time being, but he returned to them as assistant operating manager, Pittsburgh Division, during the period 1905 to 1911. In the latter year he became operating manager of the Pittsburgh Division. He was appointed assistant to the President in 1917, a position which he held until 1927, when he was made assistant general operating manager, acting in that capacity until 1931. During that year he was again appointed assistant to the President, which position he filled until his retirement on June 1, 1940. He thus completed a service record with the American Bridge Company, and its predecessor, of more than forty-two years and a very active business career of forty-five years.

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[1] Memoir prepared by C. F. Goodrich, M. Am. Soc. C. E.

Mr. Williams' remarkable ability as a manager of production operations was due not only to his engineering education and experience, but to his remarkable memory of names, personalities, and capabilities. During his management career there was scarcely an employee of the American Bridge Company of a few years' service whom he could not call by name, and many were indebted to him for encouragement and personal aid in adversity. His cheerfulness and courage were always present, and he had a way of belittling difficulties and solving them with his store of common sense that was infectious. He was a large man, of impressive personality, so that his presence in any gathering, whether business or social, was always immediately felt.

In July, 1936, the United States Steel Corporation began the publication of the company magazine, *U. S. Steel News.* Mr. Williams was an associate editor of this magazine until his retirement and contributed many articles to its pages. Just before his retirement he wrote a history of the American Bridge Company.

Until care of his health prevented, Mr. Williams was an ardent golfer and a member of Stanton Heights Golf Club, in Pittsburgh. His main hobby was the study of the various breeds and characteristics of cattle and horses. He followed, by means of published journals and acquaintance with owners, the successes in raising and the sales of all the famous thoroughbred herds of cattle in the United States and could recite from memory the pedigree of most of the famous race horses for more than one generation. One of his greatest delights was to collect two or three of his friends and attend the trotting races at Lexington, Ky., not for the races primarily, but to examine and admire the horses and visit nearby breeding stables.

On October 15, 1907, Mr. Williams was married to Nellie Steck Lloyd who died on May 24, 1918. He is survived by two daughters, Elizabeth (Mrs. John F. Tim, Jr.) and Martha (Mrs. W. Kirk Gilmore); two brothers, Carlton Williams and Harris Williams; and one sister, Mrs. Myra W. Cline.

He was a member of the Engineers' Society of Western Pennsylvania, the American Welding Society, the American Iron and Steel Institute, and the American Institute of Steel Construction.

Mr. Williams was elected a Member of the American Society of Civil Engineers on June 24, 1914.

CHARLES POPE YEATMAN, M. Am. Soc. C. E.[1]

DIED JUNE 23, 1939

Charles Pope Yeatman, the son of Walker Meredith and Eva (Ammen) Yeatman, was born on January 30, 1853, in Cincinnati, Ohio. He received his education at Hughes High School, of that city, and was awarded a silver medal for excellence in mathematics.

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[1] Memoir prepared by W. C. Yeatman, Los Angeles, Calif.

He started working in September, 1872, as an assistant to Joseph Earnshaw, a civil engineer of Cincinnati. He stayed there for three months. Then, from June, 1873, to February, 1877, he was in the employ of the Cincinnati Southern Railway. He started as rodman and later became resident engineer. From May to November, 1877, he was resident engineer with the Cincinnati and Portsmouth Railroad Company.

He resigned from this company to take a position in Colombia, South America, a step that was to have a great effect on his future, as the greater part of his engineering and business career was in that country. He soon learned to speak Spanish as fluently as English, and his intimate knowledge of the country, its people and customs, made a special demand for his services in Colombia.

From November, 1877, to August, 1883, he was with the Antioquia Railway, first as assistant engineer, and later as chief engineer. This railroad was surveyed from Puerto Berrio, on the Magdalena River, to Medellin, capital of the Department of Antioquia, but was only partly completed when work was temporarily abandoned in 1883, due to lack of funds. On cessation of this work he returned to the United States, and from February to September, 1884, he was in charge of a survey party on extensions of the Norfolk and Western Railway Company.

Shortly after the termination of this work he was again called to Colombia as chief engineer of the Girardot Railroad, serving from November, 1884, to October, 1886. He made a survey for an extension of the short railroad then running a few miles out of Girardot, to Bogotá, the capital of Colombia. This country was extremely rugged, and changed in altitude from a few hundred feet at Girardot to 9,000 ft near Bogotá.

Returning to the United States, he was resident engineer of the Louisville Southern Railroad from February, 1887, to July, 1888, while this road was being built from Louisville to Harrodsburg, Ky. On the completion of this work he became assistant engineer on construction of a reservoir for the water works of Covington, Ky., this work lasting from July, 1888, to January of the following year.

From March, 1889, to February, 1890, he was engineer for Sloss Iron and Steel Company at Coalburg, Ala., where this company then had extensive coal mining and coke operations. From July, 1890, to May, 1891, he was locating engineer for Allison Shafer and Company, near Morristown, Tenn.

From May, 1891, to November of that year, he was federal inspector of dike at Rising Sun, Ind. From November, 1891, to April, 1892, he was federal inspector of levee at Shawneetown, Ill. These were both navigation improvements on the Ohio River.

On completion of this work he was again called to Colombia, South America, where he was chief engineer of the Cucuta Railroad from May, 1892, to November, 1894. During this employment he had two attacks of yellow fever, which was prevalent at Cucuta during that period. When he recovered, local doctors considered him quite fortunate, stating that the chance of recovering from a second or recurring attack was about one out of five hundred.

He returned to the United States to regain his health, and in December, 1895, again returned to Colombia, where he remained until he retired in April, 1911. During the first three years he was assistant manager of the Barranquilla Railroad and Pier Company, and during the remainder of the time general manager of the company. This was an English firm, whose property consisted of an ocean pier 4,000 ft long, and a short railroad connecting with Magdalena River steamboats at Barranquilla. It was of great importance to the country, as it handled a large percentage of the imports and exports.

In this last position his relations with the Board of Directors in London, England, were most cordial, and in appreciation of his untiring efforts in behalf of the company they voted him a substantial bonus when he retired. In 1918 they cabled him from London to go back to Barranquilla and straighten out some difficulties of management that they were having. This was a matter of several months, and was his last active work.

He wrote articles for *The Engineering Magazine* in 1893 and 1897. He lived a long and useful life, and was unflagging in his devotion to his work. His loyalty, scrupulous honesty, and ability were appreciated by all who knew him.

He was married to Sarah K. Clark on December 28, 1876. They had one son, Walter C. Yeatman, who survives him.

Mr. Yeatman was elected a Member of the American Society of Civil Engineers on February 2, 1887.

CLAUD FRANCIS BLAIN, Assoc. M. Am. Soc. C. E.[1]

Died February 14, 1940

Claud Francis Blain was born in Alexandra, Victoria, Australia, on March 13, 1872.

He served his apprenticeship in the office of F. I. Byerley, civil engineer, and was graduated at the age of twenty. He held the licensed surveyor's certificate in both New South Wales and Western Australia, and also the shire engineer's certificate. For four years after his graduation he was assistant to Mr. Byerley and the following year became a member of the firm. Mr. Blain was a partner of Byerley and Blain, Civil Engineers and Surveyors, in Rockhampton, Queensland, from 1897 to 1904. For the succeeding four years he was in private practice at Perth, Western Australia, and Brisbane, Queensland.

Mr. Blain joined the Department of Public Works of New South Wales on January 5, 1909, in the capacity of location engineer, and for fourteen years he was engaged on the location and layout of storm water sewers and water supply pipe lines. This work included the selection and location of sites for dams for water storage in country towns.

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[1] Memoir prepared by W. H. Roper, Esq.

On July 1, 1923, Mr. Blain was appointed to the position of supervising engineer in the Department of Public Works and was in charge of investigation of water supplies. His work included the office and field investigation for new projects, and for augmentations of existing installations, together with the direction of surveys, preparation of estimates, and submission of reports to the chief engineer or to Parliamentary Standing Committees for Public Works.

Some of the more important water supply works in New South Wales that Mr. Blain dealt with in his official capacity included the Chichester River gravitation pipe line for Newcastle; Umberumberka Dam and pipe line for Broken Hill; Winburndale Dam and gravitation main as an augmentation for Bathurst; South West Tablelands District scheme; Taree-Wingham pumping project from Dingo Creek; Puddledock Dam and gravitation main for augmenting Armidale water supply; and augmentations for Goulburn, Lismore, Lithgow, Mudgee, Orange, Blayney, Albury, Parkes, and Katoomba. The latter two are of special interest in that each involves an earth dam about 80 ft high, impounding a storage of 400 million gal, and incorporating a reinforced-concrete core wall—a type of design now obsolescent. These dams are the largest earth embankments in New South Wales.

Mr. Blain made several investigations for district proposals that had not reached the construction stage at his retirement. In this category comes the Southern Riverina scheme, which will serve three towns—Lockhart, The Rock, and Henty; also, two villages—Yerong Creek and Uranquinty.

The source of supply is the Murrumbidgee River at Wagga Wagga, where the water would be pumped and treated. The Coolamon-Ganmain is another such proposal, for which the point of supply has been altered so that, instead of pumping from the Murrumbidgee River south of Ganmain, the intake will be at Jugiong, as service has been arranged as an extension of the South West Tablelands undertaking.

To broaden his knowledge of water supply engineering, Mr. Blain studied advancements in other states and overseas. He submitted reports in this connection on the Wimmera-Mallee domestic and stock water supply system in Victoria, and on the progress of earth-dam construction there.

At a time when Australia was not erosion-conscious to the extent now obtaining, Mr. Blain's foresight and observation were such that he visualized the disastrous consequences of the indiscriminate destruction of arboreal cover on steep country and on land of an erodible character. He noted the rate of siltation of dams, and the increasing turbidity of streams following settlement of catchment areas, and took an active part in endeavoring to restrict conditions of leaseholds on these lands to check the menace of erosion. As a member of the Erosion Committee, he impressed his attitude on other members and induced the Committee to adopt firm measures in handling these matters.

Mr. Blain's duties necessarily involved much country traveling and contact with local government bodies in the state. He possessed a fund of anecdotes and a dry humor that won many friends for him among the Councils, and his sound knowledge inspired them with confidence in his judgment and advice. The early surveying experience and discipline inculcated by his rigorous training were reflected in the rapid rate at which he traversed country on reconnaissance and in his conscientious attention to the work throughout the entire period of these visits.

On his retirement from the Service at the age of 65, Mr. Blain went to Melbourne, Victoria, where he had a large circle of acquaintances. He joined two separate bowling clubs, and soon became proficient at the game. His wife suffered ill health shortly after settling in Melbourne, and some months later Mr. Blain underwent an operation in South Melbourne Hospital because of complications resulting from an old injury sustained when in Rockhampton. This left him unable to leave his residence, and his condition became gradually worse, culminating in his death at Hawthorn (Melbourne) on February 14, 1940. Mrs. Blain had died on December 21, 1939.

Mr. Blain was elected an Associate Member of the American Society of Civil Engineers on April 3, 1922.

ARTHUR WILLIAM PAUL BLOHM, Assoc. M. Am. Soc. C. E.[1]

DIED JUNE 19, 1940

The untimely death of Arthur William Paul Blohm on June 19, 1940, was a deep loss to his friends and associates and also terminated the work of a valuable and respected engineer.

Mr. Blohm, the son of William H. F. and Lena (Deininger) Blohm, was born on October 21, 1899, in Baltimore, Md., where he resided during his entire lifetime. He was educated at the Baltimore Polytechnic Institute, and was graduated in 1919. Later he supplemented his education at the Johns Hopkins University, in Baltimore, where he pursued courses in chemistry; in Cincinnati, Ohio, where he took a course in stream pollution; and in Washington, D. C., where he took a course in industrial hygiene. Both of the latter courses were under the direction of the United States Public Health Service.

In March, 1919, he was appointed engineering assistant in the Bureau of Sanitary Engineering of the Maryland State Department of Health. In March, 1923, he was advanced to assistant sanitary engineer, and on May 1, 1940, six weeks prior to his death, he was promoted to senior assistant sanitary engineer. During his twenty-one years of service with the Department of Health his work covered nearly every phase of the field of public health engineering.

Mr. Blohm rendered valuable service to the State during the record flood on the Potomac River, in 1936, while in charge of flood rehabilitation work at Hancock, Md., and in a consulting capacity at Williamsport, Md. In 1937, as a collaborating sanitary engineer with the U. S. Public Health Service during the disastrous floods in the Ohio River Valley, he was in charge of flood rehabilitation work in the area surrounding Huntington, W. Va. In 1936 he assisted in a training program for sanitarians for the full-time county health departments of Maryland, giving instruction in water supply and sewage treatment. During that same year he was assistant field director of an occupational disease study made in Maryland by the U. S. Public Health Service.

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[1] Memoir prepared by a Committee of the Maryland Section consisting of George L. Hall and James R. McComas, Members, Am. Soc. C. E., and Lloyd C. MacMurray, Jun. Am. Soc. C. E.

Mr. Blohm was a member of the American Water Works Association, American Public Health Association, Federation of Sewage Works Associations, and the Maryland-Delaware Water and Sewerage Association. He was secretary of the latter from 1930 to 1934 and secretary-treasurer from 1934 until his death. He served from 1929 to 1935 as a member of the abstract committee of the American Water Works Association and was active in the Four States Section of that association.

Mr. Blohm's work for the Maryland-Delaware Water and Sewerage Association was outstanding and to a large degree responsible for the high standards and success of the association. In addition to his duties as secretary-treasurer, he prepared several papers on technical subjects for the annual conferences of the association and served as an instructor at each of the Short Schools held for water and sewage treatment plant operators. This untiring work for the association gained for him wide recognition for his personal qualities of loyalty, intelligence, and professional integrity. Among his foremost qualities were his modesty and unfailing kindness and courtesy to acquaintances, friends, and associates.

Mr. Blohm was elected an Associate Member of the American Society of Civil Engineers on July 12, 1937.

ERNEST WILLARD CRAWLEY, Assoc. M. Am. Soc. C. E.[1]

DIED APRIL 18, 1940

Ernest Willard Crawley was born on September 25, 1879, in Warren, R. I., of staunch New England parentage. His parents were William H. Crawley and Emma L. Bowen. His early education was obtained in the public schools, and he qualified in 1897 for entrance into Brown University at Providence, R. I., from which he was graduated in 1901 with the degree of Bachelor of Science in Civil Engineering. He was a member of the Delta Tau Delta Fraternity and was given signal honors in science by Sigma Xi.

In July, 1901, Mr. Crawley began his professional career in the Engineering Department of the New York, New Haven and Hartford Railroad Company on masonry construction and four-track grade crossing elimination at Bridgeport, Conn.

In August, 1902, he accepted a Civil Service appointment as inspector in the War Department, U. S. Engineers, and was first assigned to the New Haven

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[1] Memoir prepared by Henry A. Whitcomb, Assoc. M. Am. Soc. C. E.

(Conn.) Suboffice of the New London (Conn.) District. Here he remained until 1916, when he was transferred as a junior engineer to the New London District Office. At the outbreak of World War I he was made principal civilian engineer in charge of river and harbor maintenance and improvement work in the New London District, replacing George E. Verrill, M. Am. Soc. C. E., who had entered the military services of the United States. Mr. Crawley held this post until the consolidation of the New London and Newport (R. I.) districts into the Providence (R. I.) District on January 1, 1920. From January, 1920, until his death on April 18, 1940, he was principal civilian assistant to the district engineer, in charge of the River and Harbor Division of the Providence Engineer District.

During his long service with the Corps of Engineers, from inspector to senior engineer, he was engaged on river and harbor improvement works from Chatham, Mass., on the east, to Greenwich, Conn., on the west, including the watersheds of the Taunton, Blackstone, Thames, Connecticut, and Housatonic rivers. He was a recognized authority on all navigation projects.

The love of the arts, literature, and music was a dominant characteristic of Mr. Crawley's mind. His friends would feel that his story was incomplete if it did not emphasize his appreciation and understanding of music, particularly the works of Wagner, Beethoven, and Brahms.

He was a thoroughly competent engineer, resourceful and energetic. His integrity was such that he would not tolerate the slightest deviation from the truth as he saw it. He was known and respected for his ability as an engineer, but his capacity for friendship was so rare and fine a thing that it seems impossible to write of him without trying to convey some appreciation of his personality.

Quiet and unassuming in manner, and highly conscientious, Mr. Crawley possessed a sense of humor and understanding that will long be remembered by his friends and associates. He was absolutely reliable in everything he said and did, true and devoted to his family, his friends, his country, and his church, as well as to the engineering profession and the Society of which he was so proud. The men who worked for him loved him, "swore by him," worked overtime for him, and would fight for him if necessary. He was happiest when helping in the improvement and the progress of his subordinates. Probably his outstanding characteristic was his innate interest in and affection for people.

Mr. Crawley's frequent advice was to solve the main problem in a broad way in advance without cluttering the mind with too many details. This practice he followed. His scientific trend of mind enabled him to coordinate men, materials, and equipment to obtain maximum results with minimum effort and waste. These qualities, together with his ability to obtain loyalty and cooperation from employees through his sympathy and understanding, made him an exceptional executive.

On October 28, 1903, Mr. Crawley was married to Sarah E. Batchelor of Warren, who, with a son, Willard, and a daughter, Elizabeth, survives him.

Mr. Crawley was elected a Junior of the American Society of Civil Engineers on May 31, 1904, and an Associate Member on September 6, 1910.

JOHN CLAIR FOWLER, Assoc. M. Am. Soc. C. E.[1]

DIED AUGUST 19, 1940

John Clair Fowler was born at Boulder, Colo., on January 5, 1878, the son of John C. and Mary C. (Dwyer) Fowler. His father died before his fifth birthday and his mother taught for many years in the schools of Boulder to educate her family.

He was educated in the public schools of Colorado, was graduated from the State Preparatory School, and received his Bachelor of Science degree in Civil Engineering from the University of Colorado, at Boulder, in January, 1903.

He began his engineering career in Gilpin County, Colorado, in partnership with Sam A. Rank, a well-known mining engineer of that part of the state. Later Mr. Rank retired, and Mr. Fowler maintained the office until March, 1909.

While living in Central City, the county-seat of Gilpin County, he served two terms as county engineer and two terms as city engineer of Central City, then a thriving mining community. He was also a deputy United States mineral surveyor while practicing in Colorado.

In 1909 he came to Spokane, Wash., and on March 3 of that year he entered the employ of the city engineer as levelman. On July 1 he was promoted to assistant engineer because of efficiency. On June 1, 1910, he was promoted to district engineer, in which position he had charge of preliminary and construction surveys for his district, as well as the supervision over construction. At that time the city was divided into three districts and each district engineer had full charge of the work in his area.

On November 11, 1917, he was made principal assistant city engineer, and on September 3, 1919, was given the position of superintendent and engineer of the Municipal Paving Plant. His ten years' previous experience in the engineering department where he passed upon the quality of paving as an engineer on construction qualified him particularly well to manage the actual production as superintendent. He served in this capacity for eleven years, during which time he built up an organization outstanding for its energy and efficiency.

He was thoroughly grounded in engineering principles and never sacrificed quality of production for speed, but secured speed of production by organization and efficiency without sacrificing quality.

On February 16, 1930, he returned to the engineering department as chief field engineer, where he was employed at the time of his death.

He was a tireless worker, never sparing himself or exacting from those in his charge anything that he himself would not do. He possessed a marked degree of ability in dispensing a fair and impartial justice in his dealings with contractors and engineers alike.

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[1] Memoir prepared by Alfred D. Butler, M. Am. Soc. C. E.

During his employment in the City Engineer's office, he had charge of construction of bridges, as well as sewer and street improvements of every character. Regardless of the type of work he had in his charge it was all subjected to the same careful engineering scrutiny, and his employer could be assured that the terms of the plans and specifications were fully met and the work done in accordance with the best engineering practices.

One of his habits was the free use of logarithms in the ordinary calculations involved in his daily work, and a familiar sight on his desk was a well-worn volume of seven-place logarithm tables.

He was married on September 5, 1906, to Alice Williams of Central City, who survives him. He leaves two daughters: Helen Pearle (Mrs. V. E. Sharp) and Alice Coleen (Mrs. H. R. Stinson). He is also survived by one sister, Pearle Fowler, a teacher.

Mr. Fowler united with the Presbyterian Church of Boulder while a student at the University and was an active church worker all his life. He was ordained an elder in the Presbyterian Church of Central City and, since living in Spokane, had served the Knox Presbyterian Church in that capacity for more than twenty-five years. He was the senior elder in point of service at the time of his death.

Mr. Fowler was elected an Associate Member of the American Society of Civil Engineers on May 13, 1918.

WILLIAM RICHARD GELSTON, Assoc. M. Am. Soc. C. E.[1]

DIED SEPTEMBER 27, 1940

William Richard Gelston was born on a farm near Elk City, Nebr., on January 1, 1868, the son of George Wells and Mary (Wilkening) Gelston. He was third in a family of nine children, all of whom survive him.

In his youth he worked on the farm of his father, taking advantage of what school opportunities the frontier community afforded. At the age of nineteen he entered Grinnell College, at Grinnell, Iowa, from which he was graduated with the degree of Bachelor of Philosophy in 1894, after working his way through the college course. He then returned to the farm until his father's death in 1899.

His engineering career began when he left the farm to join a railroad location crew as axman, and for the next eight years he was engaged in the extension of the Burlington lines through the western and midwestern states. Promotion came rapidly to division engineer in charge of construction and then locating engineer. Within this time he was engaged in railway location between Muscatine, Iowa, and Peoria, Ill.; an extension from Centralia, Ill., to Herrin, Ill.; he acted as superintendent of construction on the Old Monroe-

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[1] Memoir prepared by Charles B. Burdick and Paul Hansen, Members, Am. Soc. C. E.

Mexico branch; and he returned to the Far West as locating engineer on a line between Orin and Worland, Wyo. His railroad work, all for the Burlington system, included an engagement as office engineer at St. Louis, Mo.

Mr. Gelston's outstanding accomplishment was his long and successful engagement as superintendent of the Citizens Water Company at Quincy, Ill., and its successor, the Quincy Municipal Water Works, which he had served for thirty-three years at the time of his sudden death. He had been in complete charge of the property under the Water Board, and, with the advice of consulting engineers, in some instances planned and executed all extensions and improvements, in addition to his duties of management. This, his principal life work, was taken up immediately upon his resignation from railroad service on January 1, 1907.

One of his accomplishments was improving and enlarging an underbuilt plant of Quincy, thus making it one of the best plants in the state. These improvements included the replacement of one of the earlier mechanical water filtration plants with a new and enlarged plant; the substitution of electricity for steam pumping after the availability of low-priced hydro power; the adoption of water softening as an addition to the service rendered; the protection of the water purity by covering the large elevated storage reservoir; the material extension of the water pipe system, including additional large feeders to supply more water at adequate pressure for present requirements and for future growth; and the rebuilding of the intakes drawing water from the Mississippi River. The property was purchased by the city and the purchase debt was entirely liquidated from the earnings. This was accomplished under a pre-war schedule of water rates inherited with the purchased property. One of Mr. Gelston's last acts was to institute a study of water rates in the anticipation of a reduction in water charges upon the completion of certain improvements then in progress.

To one familiar with the vicissitudes of municipal undertakings, three decades in the management of one property is no mean accomplishment. Success requires good management, sound policies, good engineering, and successful public relations. Mr. Gelston possessed the qualities required in a high degree—he was not only "big" in stature. This quality extended to his every-day task. He was industrious, thoughtful, patient, efficient, tactful, and far-seeing in all that he undertook. He worked on a plan, well conceived and efficiently executed. He had a personality that carried conviction to those with whom he had business relations. Few men have the character, ability, personality, and knowledge for these accomplishments; some of these attributes were inherited, and some were acquired under adverse circumstances.

He had a wide acquaintance through his membership in the American Water Works Association, for which he served as Director in 1934–1935, the Illinois Society of Engineers, the Quincy Chamber of Commerce, and the Quincy Rotary Club, of which he was Past-President. He was a progressive member of his profession, and the author of several papers on water-works subjects.

On February 11, 1909, he was married to Elizabeth Chavey, who survives him. Also surviving are his two sons, Richard and George Frederick Gelston. Richard Gelston assisted his father for several years as superintendent of the Quincy Water Purification Plant and has succeeded him in the management of the property.

Mr. Gelston was elected an Associate Member of the American Society of Civil Engineers on September 12, 1916.

ANDREW HUGH GREEN, Assoc. M. Am. Soc. C. E.[1]

Died December 29, 1939

Andrew Hugh Green was born in Chicago, Ill., on November 7, 1869, the only son of Oliver Bourne Green and Emily Louise (Pomeroy) Green. He received his education at Harvard University, in Cambridge, Mass., graduating with the degree of Bachelor of Arts in 1892. In 1896 he received the degree of Bachelor of Science in Civil Engineering from Massachusetts Institute of Technology, also in Cambridge.

In the summer of 1891 he had charge of the plant of Green's Dredging Company, of which his father was the owner, engaged in operations upon the site of the World's Columbian Exposition at Chicago. After spending the winters of 1892 and 1893 in Europe he returned to Chicago and designed and superintended the construction of several wooden barges of 500 to 600 tons capacity.

He was a student at Massachusetts Institute of Technology in 1895 and 1896, during which time he acted as consultant for the construction of a new plant for Green's Dredging Company. During four months of each year he was in charge of the design and execution of this work. From June, 1896, to January, 1901, he was in direct charge of the plant and operations of Green's Dredging Company in Chicago. He designed and constructed the first steel hopper-bottom dump-scows in use in Chicago and the first turntable pile driver. He was also the first designer to mount the A frame of dipper dredge on the spud casings.

His father retired in 1899 and Andrew Green became head of Green's Dredging Company. He sold the firm to the Lydon and Drews Company in 1901 and both companies afterward were merged with the Great Lakes Dredge and Dock Company. He then became an assistant engineer for the Chicago, Milwaukee and St. Paul Railroad Company.

Shortly thereafter Mr. Green purchased a plantation of about 1,000 acres on the island of Dominica, British West Indies, and went there to live. The principal product of the plantation was lime juice which he shipped to England for use in the manufacture of dyes. He also produced vanilla beans and rum in commercial quantities. He built a cane mill, power plant, and warehouses on the plantation in addition to an ingenious and unique tropical residence. Hurricanes several times unroofed his home and plantation buildings and he learned to build them so that the metal roofs could be retrieved and readily restored after the hurricane had passed.

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[1] Memoir prepared by Robert Isham Randolph, M. Am. Soc. C. E.

He was also much interested in the study and prevention of flood damage in the water courses on the island which were subject to sudden rises from heavy tropical rains, and he experimented with rock-filled wire sausages as baffles and with concrete equilateral trilons used as retards which always landed right-side-up no matter how often the current turned them over.

Mr. Green traveled extensively and usually made one trip to Chicago every year. At these times it was the writer's pleasure to visit with him and compare notes on engineering and other affairs at home and abroad. He was a large, fine figure of a man with a very attractive personality, and his keen mind covered a wide range of interests. He had survived most of his early business associates who, in the memory of this writer, always spoke of him with the same admiration and affection that the writer felt for him.

Mr. Green was elected an Associate Member of the American Society of Civil Engineers on October 2, 1901.

FRANK HOLMES, Assoc. M. Am. Soc. C. E.[1]

Died July 10, 1940

Frank Holmes, the son of William and Maria (Edwards) Holmes, was born on October 28, 1871, at Spondon, Derbyshire, England.

In September, 1872, Mr. Holmes came to the United States with his parents who established residence in Chicago, Ill. He was educated in the public grade schools and attended night school to complete his engineering education. In 1895 he started night courses at Jenner Medical College, in Chicago, and received the degree of Doctor of Medicine and Surgery in June, 1899.

Mr. Holmes' education was enriched and enlarged through an intense thirst for knowledge, which led him at all times, and everywhere throughout his life, to seek information beyond that applicable to, or offered by, the work on which he was immediately engaged. Therefore, he was never at a standstill and content with present attainments, but reached forward to new acquirements in order that he might better perform the duty of today and be prepared for that to which he might be called tomorrow. In consequence, the positions which he filled have been important and in each he did his work patiently, carefully, earnestly, and conscientiously, faithful always to the charge committed to his trust. The record of his services that follows, covering an active professional life, is one in which he rightfully took pride.

He began his engineering work with the firm of Winslow Brothers Company, Ornamental Iron Contractors, in Chicago. In 1891 Mr. Holmes was employed by G. L. Clausen, consulting civil engineer and surveyor, of Chicago, during which time he supervised the construction of sewers, water lines, pumping stations, wells, and street and sidewalk improvements.

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[1] Memoir prepared by Clarence W. Anderson, Esq., Chicago, Ill.

In 1896 Mr. Holmes entered the service of the Engineering Department of the City of Chicago as assistant engineer on the $10,000,000 Northwest Land Tunnel for water distribution. In 1902 he entered the services of D. H. Burnham and Company, as structural engineer, and worked on such structures as the Frick Annex, Pittsburgh, Pa.; Hibernia Bank Building, New Orleans, La.; John Wanamaker Stores, New York, N. Y., and Philadelphia, Pa.; and the Union Station at Washington, D. C. Mr. Holmes was architect's representative in charge of the construction of foundations for the first section of the Wanamaker Store in Philadelphia and also for the foundations and superstructure of the Union Station, in Washington, D. C.

Mr. Holmes entered the services of Thompson-Starrett Company, Builders, in New York, N. Y., in 1906, as superintendent of construction. He supervised the construction of such structures as the Union Station, in Washington, D. C.; and the Wanamaker Building, Stock Quotation Building, 123 William Street Building, Baker Building, the third section of the Stearn Store on 23d Street, and the Woolworth Tower Building at Broadway and Bleeker Street, all in New York City.

In 1913 Mr. Holmes became resident engineer, Permanent Building Division, Panama Canal, Canal Zone, Panama. He was in charge of design, location, and construction of permanent buildings, and the transfer and erection of temporary buildings for the care and housing of canal employees. Permanent buildings included the Administration Building; sixty, two, and four family houses; one school at Balboa Heights; Hydroelectric Station at Gatun Dam; four large substations between Gatun and Balboa; shops and office buildings in Balboa; two large commissary store buildings; one refrigeration plant; and three radio stations. Mr. Holmes returned to Thompson-Starrett Company in 1915 as general superintendent in charge of the erection of the Chamber of Commerce Building and Grand Theater in Pittsburgh, and many other structures.

In 1918 he became superintendent of construction and branch office manager at Pittsburgh for the George A. Fuller Company, Builders, of New York City. He became general superintendent of construction for the W. T. Grange Construction Company in Pittsburgh in 1921.

From 1923 to 1931 Mr. Holmes was superintendent of construction for Starrett Brothers, Incorporated, of New York City and Chicago. In 1933 he became director of research with the firm of H. M. Preston Company and was connected with them until a short time before his death.

He passed away on July 10, 1940, and is survived by his widow, Lillian, to whom he was married in December, 1927. He is also survived by a sister, Mrs. Emma Sickle. "Doc," as every one called him, had a large circle of friends who will miss him greatly.

Mr. Holmes was elected an Associate Member of the American Society of Civil Engineers on December 1, 1908.

LOUIE THEODORE JESSUP, Assoc. M. Am. Soc. C. E.[1]

DIED OCTOBER 1, 1939

In the untimely death of Louie Theodore Jessup at the age of fifty, the civil engineering profession and the federal government lost an engineer of outstanding ability and a servant of unswerving fairness, integrity, and honesty of purpose. He was born in Latah, Wash., on January 21, 1889, the son of Charles Wesley Jessup and Zena Agnes (Gallaher) Jessup, whose ancestry on his father's side can be traced to Bromhill, England, where records of the family are stored in a church built in 1100. Some of the family came to America in 1772. His mother descended from Scotch-Irish stock. Her parents were early pioneers in Walla Walla, Wash.

The early life of Louie Theodore Jessup was spent on a wheat farm in northern Idaho under conditions that developed self-reliance and independence of thought and action. These traits characterized his life's work. Following his education in the public schools of Kendrick, Idaho, he entered the University of Idaho at Moscow, Idaho, from which he was graduated in 1913 with the degree of Bachelor of Science in Civil Engineering. As an undergraduate at the university, he was an active participant in campus affairs, and a leader in athletic activities and in his college fraternity, Beta Theta Pi.

Except for minor civilian engineering services prior to and shortly after completing his college course, Mr. Jessup's entire career as an engineer was spent in government service. His career started in September, 1913, with an appointment as junior drainage engineer in the Division of Drainage Investigations, U. S. Department of Agriculture. It ended, after twenty-six years' service, in the same government department, during which he was advanced to the grade of drainage engineer with outstanding accomplishments in this field of engineering.

During his first ten years as a government engineer, Mr. Jessup was occupied principally on research work. He conducted numerous investigations and experiments pertaining to the drainage and reclamation of agricultural lands in Colorado, Nebraska, Idaho, and Washington. He made studies to determine the proper depth of drains for waterlogged irrigated lands, the effectiveness of drainage by pumping from wells, methods of draining irrigated shale lands, the discharge of drains serving irrigated lands, methods of reclaiming alkali lands, and the proper design of structures used in draining agricultural land. His findings on these and allied subjects are important contributions to engineering knowledge and are published in several government bulletins and numerous professional periodicals.

The later years of Mr. Jessup's professional career were devoted less to research and more to service work. He was called upon to determine and recommend plans for effective drainage on many reclamation projects throughout the Western States. He served importantly as consulting engineer for the Bureau of Biological Survey in the design of reflooding works for Lower Klamath Lake, California, and Klamath Marsh and Malheur Lake, Oregon. On the Malheur Lake project he distinguished himself as an expert investigator and witness in behalf of the government's claim to the waters of Harney Basin for the Malheur Bird Reserve. For several years prior to his death Mr. Jessup conducted involved field studies in Kootenai Valley, Idaho, pertaining to the effect of water elevations in Kootenai River on adjacent agricultural lands, as influenced by backwater from projects in British Columbia. This work was performed in cooperation with the U. S. Geological Survey in connection with the intensive international investigations which are conducted for the benefit of the U. S. Department of State and the International Joint Commission. His work on this complicated problem was outstanding. He served ably in that capacity for several years.

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[1] Memoir prepared by C. G. Paulsen, M. Am. Soc. C. E.

During the last three years of his life, Mr. Jessup filled an important place in the interstate snow survey and water supply forecast program which the federal government sponsors throughout Western United States. He was largely responsible for this program in Washington, northern Idaho, and western Montana. He also served several years as a member of the Northwest Planning Board.

Mr. Jessup was a member of the American Society of Agricultural Engineers and of the American Geophysical Union. He held a license to practice civil engineering in the States of Idaho, Washington, and Oregon. For the last twenty years of his life he made his home at Yakima, Wash.

He was married to Bertha Leighton, of Weiser, Idaho, on October 1, 1915. Mrs. Jessup and two daughters, Ethel and Mary, survive him. He was honored and loved by all of his engineering associates who reside in many states.

Mr. Jessup was elected an Associate Member of the American Society of Civil Engineers on July 6, 1920.

ROBERT JOSEPH JUSTICE, Assoc. M. Am. Soc. C. E.[1]

DIED AUGUST 12, 1939

Robert Joseph Justice was born in Burlington, Kans., on October 5, 1898. He was the only child of Joseph C. and Martha E. (Flowers) Justice. He attended grade school at Edna, Kans., and high school at Topeka Kans., from which he was graduated in January, 1917.

He joined Company A, 110th Engineers, on April 14, 1917, and went overseas where he saw service at the front. With his company he engaged in the following battles: Amiens Sector, Alsace Sector, Saint-Mihiel offensive, Meuse-Argonne offensive, and Verdun occupation. When he returned to the United States, he was mustered out of service, and received his honorable discharge on May 3, 1919.

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[1] Memoir prepared by R. H. Pennartz, Assoc. M. Am. Soc. C. E.

The following fall he entered the Engineering School of the University of Kansas, at Lawrence, Kans., where he completed his education, specializing in highway engineering.

Mr. Justice rose from the position of inspector with the Kansas Highway Commission to that of assistant division engineer at Hutchinson, Kans. In February, 1926, he resigned as assistant division engineer and accepted a position as county engineer of Harvey County, Kansas, and resident engineer for the Kansas Highway Commission. During this time he not only attended to the duties of the county engineer but also took charge of all state and federal aid construction in this county. In October, 1928, he became ill and resigned to enter the Veteran's Hospital at Fort Lyon, Colo.

After his recovery he was called to Topeka by the Kansas Highway Commission and placed in charge of some of the major construction projects for the state. The last large project of which he was in charge was the $1,500,000 bridge and viaduct across the Kansas River at Topeka, which will stand as a lasting monument to his ability.

His research work in soil compaction, sand gradations, and concrete mixtures is most outstanding, and many of his conclusions have been accepted by the engineering profession and are incorporated in many state and federal specifications. His research in concrete mixes resulted in his election to the National Highway Research Board. His paper on "Vibrated Concrete" was published in the Kansas Engineering Society's *Transactions* of 1938 and received wide comment and praise.

Mr. Justice was quiet and unassuming but he had a keen sense of humor. His dry wit was always a source of amusement to his friends but, since he was never unkind, he never used it at the expense of others.

He was a member of the Kansas Engineering Society, the National Highway Research Board, the American Legion, and the Veterans of Foreign Wars.

On October 15, 1924, he was married to Eva Jones of Topeka, and his home life was supremely happy. His widow and his son, Robert Joseph, Jr., survive him. In his private life he was kindly, sincere, and honorable—a good companion and a devoted husband and father.

Mr. Justice was elected an Associate Member of the American Society of Civil Engineers on May 25, 1931.

HARRY KORNFELD, Assoc. M. Am. Soc. C. E.[1]

DIED NOVEMBER 8, 1934

Harry Kornfeld was born in New York, N. Y., on September 23, 1887, the son of Julius and Eliza (Simon) Kornfeld. He was educated in the New York City public schools and the Townsend Harris High School in Manhattan. In 1908 he was graduated from the College of the City of New York with the degree of Bachelor of Science, cum laude. He later received the degree of Civil Engineer from Cornell University, at Ithaca, N. Y.

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[1] Memoir prepared by Alfred E. Kornfeld, Affiliate Am. Soc. C. E.

From 1911 to 1914 he worked with the late Hugh L. Cooper,[2] M. Am. Soc. C. E., a consulting engineer of Keokuk, Iowa, on investigation, design, and construction of hydroelectric plants. One of the main projects during this period was building the Keokuk Dam for the Mississippi River Company.

In 1915 Mr. Kornfeld went to the State of Washington to do some preliminary testing for a hydroelectric plant. In October, 1915, he went to Canada (St. Joseph Dalna) to do preliminary testing for a hydroelectric plant. This project, being of international interest, was rejected by the New York State Legislature. In 1919 he went to Chile, South America, for Mr. Cooper and built an electric power plant for the Cooper mines of the Guggenheim interests.

From 1919 until his death he served as assistant engineer in the Department of Construction of the General Electric Company in Pittsfield, Mass.

Mr. Kornfeld was a member of the Cornell Society of Civil Engineers, Phi Beta Kappa, Sigma Xi honorary fraternity, and the Pittsfield Section of the American Institute of Electrical Engineers. His recreation consisted of outdoor sports.

Mr. Kornfeld was elected a Junior of the American Society of Civil Engineers on September 3, 1912, and an Associate Member on October 9, 1917.

RICHARD McCULLOCH, Assoc. M. Am. Soc. C. E.[1]

Died August 28, 1940

Richard McCulloch, born in St. Louis, Mo., on June 3, 1869, was the son of Robert and Emma (Paxton) McCulloch, both of whom were descended from Scotch families who settled in Virginia in colonial times. In the family tree appears the name of Andrew Lewis of Virginia, who served as a General in the Revolutionary War under George Washington.

Mr. McCulloch's father was a cadet at the Virginia Military Institute, at Lexington, Va., at the outbreak of the Civil War, and was one of the student body who "went out" to serve in the Confederate Army under General "Stonewall" Jackson. He participated in Pickett's famous charge at the Battle of Gettysburg, where he was twice wounded.

Richard McCulloch attended the St. Louis public schools, and after the usual high school course he entered Washington University, at St. Louis, as a student of Mining Engineering. In 1891, upon the completion of the five-year curriculum prescribed at that time, he received the degree of Mining Engineer. In recognition of his subsequent achievements, Washington University conferred upon him the honorary degree of Master of Arts in 1905.

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[2] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 103 (1938), p. 1772.

[1] Memoir prepared by A. S. Langsdorf, Dean, Schools of Eng. and Architecture, Washington Univ., St. Louis, Mo.

For a short time after his graduation from college, Mr. McCulloch served as assistant geologist in the U. S. Geological Survey in Missouri, Arkansas, and Indian Territory; thereafter, for about a year, he was employed as chemist and assayer at the plant of the Great Mexican National Smelting Company at Catorce, Mexico. During part of 1893 he was employed at the plant of the General Electric Company in Schenectady, N. Y. Thereafter, from 1893 to 1899, he served as chief engineer of the National Railway Company of St. Louis which comprised a number of street railway lines that later became component parts of the city-wide merger of traction interests. This period was one of great activity in electrifying the older horsecar and cable-car systems, and Mr. McCulloch played an active rôle in this pioneering work. Some of the work he did at that time is embodied in numerous contributions to the *Proceedings* of the American Electric Railway Association and other technical publications.

Between 1899 and 1901 Mr. McCulloch was engaged in engineering work in Europe, principally as chief engineer of the Geneva Tramways Company in Geneva, Switzerland. One of his favorite stories about his experiences abroad poked fun at his initial attempts to learn to speak French; he had taken up residence at a pension for the sake of the conversational practice with the other guests, and one evening, on sitting down at the table, he decided to start the talk by announcing "J'ai beaucoup de faim" (I am very hungry); what he actually said, to the huge amusement of the natives, was "J'ai beaucoup de femmes" (I have many women).

Returning to the United States in 1901, Mr. McCulloch became assistant general manager of the Chicago City Railway Company. In 1903 he returned to St. Louis as assistant general manager of the United Railways Company of St. Louis; later, in 1907, he became vice-president and general manager; and, in 1914, president and general manager, serving in that capacity until 1919.

In 1906 Mr. McCulloch was married to Mary Grace Beggs of Milwaukee, Wis., who, with their three children, John I. B. McCulloch, Robert Paxton McCulloch, and Mary Sue McCulloch (Mrs. Whipple Van Ness Jones), survives him. Mrs. McCulloch is the daughter of the late John I. Beggs, who was long a prominent executive in the public utility field.

Following his retirement as president of the United Railways Company of St. Louis, Mr. McCulloch served until 1925 as vice-president of the St. Louis Car Company; from 1925 to 1927 he was president of the Louisiana Pulp and Paper Company; and from 1927 until his death he was president of the Beggs Investment Company and of its numerous subsidiaries. He also served as a director of the North American Company.

In addition to his affiliation with the Society, Mr. McCulloch was a member of the American Institute of Electrical Engineers and of the Engineers' Club of St. Louis. He held membership in the Society of Colonial Wars, Sons of the Revolution, and Society of the Cincinnati. He was a Thirty-Second Degree Mason, and a member of the Knights Templar and the Shriners. He possessed an unusually genial and engaging personality which greatly endeared him to his numerous friends.

Mr. McCulloch was elected an Associate Member of the American Society of Civil Engineers on March 2, 1898.

JAY DeWITT MOORE, Assoc. M. Am. Soc. C. E.[1]

Died March 22, 1940

Jay DeWitt Moore, the son of Samuel M. and Mary A. Moore, was born at Bevier, Macon County, Mo., on January 21, 1881. His father was a native of that county, and his mother was a native of Iowa; both were of pioneer stock. When Jay Moore was three years old the family moved to Montana and there he grew to manhood. He attended, and received his diploma from, the high school at Great Falls, Mont.

When Jay Moore was in his senior year at high school the Spanish-American War broke out, and on May 1, 1898, he, with others of his class, enlisted in Company A, First Montana Infantry, and was mustered into service at Helena, Mont., on May 8. This regiment was sent to the Philippines, and Mr. Moore served there during the war with Spain and through the Aguinaldo insurrection; he was honorably discharged on October 17, 1899, at San Francisco, Calif. During his service in the Philippines he readily learned the Spanish language, which was a great help to him later in Latin America.

On his return to civil life, Mr. Moore obtained employment with the United States Bureau of Reclamation. From 1905 to 1911 he was junior engineer with that Service, first as chief of party on location of canals and laterals, and on topographic surveys; then as assistant to the division engineer on the construction of canals and headworks. A considerable part of his service with the Reclamation Service was on the Belle Fourche Project in South Dakota. From 1911 to 1912 Mr. Moore was chief of party on the location of 70 km of railroad under the Department of Public Works of the Dominican Republic. For one year, 1912–1913, he was division engineer with the Reo Medina Irrigation Company, San Antonio, Tex., and was in charge of the construction of a division of an irrigation project. In 1913 he again went to Latin America, this time to Honduras, where he served for the next three years as engineer for the Agelteca Mining Company. His work there was on the exploration of iron ore fields and investigations of hydroelectric power projects. While on this assignment he came to the United States in 1915 on a vacation. He had engaged passage on the United Fruit Company's steamer *Marawyne,* but fortunately he arrived at Belize, British Honduras, an hour too late to catch this steamer, which was lost without a trace in the Yucatan Channel.

During the year 1917–1918 Mr. Moore was employed by the Cerro de Pasco Mining Company, Peru, South America, as engineer in charge of mine surveys and track maintenance.

Prior to the outbreak of the World War Mr. Moore had made application for a commission in the Engineer Officers Reserve Corps, and when war came he was ordered to the training camp at Plattsburg, N. Y. At the conclusion of the three months' training period he was offered a lieutenant's commission, which he refused. On January 16, 1918, he accepted a commission as captain, and on April 27, 1918, was assigned to active duty at Camp Lee, Virginia, where he aided in the training of troops until the end of the war. He was honorably discharged on December 2, 1918, at Camp Leach, Maryland. On March 17, 1919, he accepted a commission as captain, Engineer Section, Officers Reserve Corps, was promoted to major on October 25, 1922, and held a commission as such until his death.

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[1] Memoir prepared by L. M. Gray, M. Am. Soc. C. E.

For the two years from 1919 to 1921 he was again in Santo Domingo, serving as resident engineer in the Department of Public Works in charge of the construction of 40 km of macadam highway. Returning to the United States in 1921, he was, for the next two years, engaged in the contracting business as a member of the firm of McDonough and Moore, building constructors of New York, N. Y.

From 1923 to 1925 he was employed by the late Col. Hugh L. Cooper,[2] M. Am. Soc. C. E., as inspector on the Wilson Dam, at Muscle Shoals, Ala., on the Tennessee River. One of his most cherished possessions was a watch given him by Colonel Cooper as a testimonial of his esteem.

In the early part of 1926 Major Moore entered the employ of Black, McKenney and Stewart, Consulting Engineers of Washington, D. C., and was sent to the Barranquilla (Colombia) office. Through this office the firm was supervising the construction of the two jetties of the Bocas de Ceniza Project at the mouth of the Magdalena River. Major Moore was chief inspector on this $10,000,000 project until the middle of 1929 when, due to the exhaustion of project funds, the work had to be suspended.

Major Moore returned to the United States from Colombia just before the beginning of the depression, and employment was difficult to obtain. From this time until the beginning of 1934 he was employed only intermittently. He worked about one year as assistant engineer for the the Department of Water Supply of the City of New York, and for a similar period as computer for the United States Coast and Geodetic Survey. In 1934 he accepted a position as resident engineer inspector with the Public Works Administration, was promoted to traveling engineer inspector in 1937, and was so employed until his death.

Major Moore was reserved and quiet, and not given to self-expression. He was a student always and his reading covered a wide range; as he possessed an exceptional memory he was unusually well informed regarding not only the developments in his own profession, but also the current national and world affairs. He was blessed with a keen sense of humor, and was a delightful companion for those who were privileged to be associated with him. He had traveled widely in foreign countries and generally far away from the tourist routes, but it was extremely difficult to persuade him to tell of his interesting experiences. His letters and reports were brief and laconic.

The outstanding feature of his engineering equipment was soundness of judgment; rarely was he wrong in his estimate of a situation or his judgment of men.

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[2] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 103 (1938), p. 1772.

Major Moore was among the finest of that restless, courageous group of engineers who are constantly seeking that "something lost behind the ranges." He was a thoroughly competent engineer, and a loyal and dependable friend; he is mourned by a host of friends.

He was married on September 3, 1934, in New York City, to Irmgard Breitenbach of Hanover, Germany, who survives him. Also surviving are a brother, Don C. D. Moore; three sisters, Isabel (Mrs. Charles Irving), Alice (Mrs. Charles Stidwell), and Dollie (Mrs. William Sanborn); and several nephews and nieces.

Major Moore was elected an Associate Member of the American Society of Civil Engineers on September 10, 1923.

FRED ELMER MURPHY, Assoc. M. Am. Soc. C. E.[1]

DIED SEPTEMBER 14, 1940

Fred Elmer Murphy was born on November 10, 1878, near Knoxville, Tenn., in the ancestral home in which he died. He was the son of William Alonzo Murphy and Ersula (Foster) Murphy. Just after the Revolutionary War, his grandfather was born in a covered wagon on the property where the family have since resided. Two of his great-grandfathers fought in the Virginia Continental Army.

Mr. Murphy was graduated from the public schools of Knox County and entered the University of Tennessee, at Knoxville, spending three years in the Department of Civil Engineering.

In 1899 he entered the service of the Cleveland, Cincinnati, Chicago and St. Louis Railway Company as instrumentman on maintenance of way at Cincinnati, Ohio, locating and inspecting structures and maintenance. He then served in 1900 as draftsman for the Cleveland Street Railway Company at Cleveland, Ohio, on the design of a power house and carbarns, and as inspector of foundations. In 1901 he became draftsman for the Variety Iron Works of Cleveland on the preparation of detail shop drawings of structural steelwork.

He was draftsman for the Tennessee Coal, Iron and Railroad Company at Ensley, Ala., designing mill buildings and preparing shop drawings for structural steelwork, from 1902 to 1905. In the latter year he became assistant engineer for the C. W. Hunt Company, in West New Brighton, N. Y., and designed structures for coal handling machinery until 1908. He then was employed as engineer for Roelker and Lee, in Richmond, Va., designing and supervising construction of various structures, mostly of reinforced concrete. Noteworthy among these were a reinforced-concrete highway bridge across the Richmond, Fredericksburg and Potomac Railroad in Richmond and a large reinforced-concrete warehouse for the Reynolds Tobacco Company in Richmond.

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[1] Memoir prepared by Charles A. Pohl, M. Am. Soc. C. E.

In 1910 Mr. Murphy became assistant chief engineer of the C. W. Hunt Company and was in charge of design of their varied line of materials, handling machinery and plants until 1916. He then was made vice-president and chief engineer and put in entire charge of engineering and production. Among the principal plants built under his direction were the Panama coaling stations at Colon, Panama, and Balboa, Canal Zone, costing about $1,000,000; the coaling station at Pearl Harbor, Hawaii, costing $150,000; and the coaling station for the Inter Island Navigation Company at Honolulu, costing $250,000.

At the outbreak of the First World War Mr. Murphy received a commission as Captain in the United States Army and was assigned to the Production Division of the Ordnance Department. His duties were largely inspection and advice in the metallurgy of ordnance material.

At the close of the war he became a partner of Frank Maloney and chief engineer of Maloney and Murphy, Engineers and Contractors of Knoxville, and was largely engaged in highway and bridge construction. In this capacity he supervised the construction of a considerable number of state highways in and around Kissimmee, Fla., and a bridge 1¼ miles long across the Peace Creek at Punta Gorda, Fla.

Mr. Murphy entered the employ of Stevens and Wood, Inc., Engineers and Constructors of New York, N. Y., in 1923, and was in direct charge of all structural engineering. The principal works which he completed for this company were: A 140,000-kw steam electric plant for the Ohio Edison Company at Toronto, Ohio, costing $20,000,000; a 10,000-kw steam electric plant at Fairbanks, Alaska, costing $2,500,000; and a 120,000-kw steam electric plant at Deepwater, N. J., costing $13,000,000. In each of these plants he was in charge of design of buildings and foundations, yard and track layout, water supply, coal handling, and coal storage. This work was completed in 1930.

In 1931 he returned to Knoxville where he lived until his death. During this time he was engaged in various capacities of a local nature, including service as engineer for the Civil Works Administration and the Knoxville Housing Authority. In February, 1940, he was appointed director of public service for the City of Knoxville, in which capacity he served until his death.

An able engineer, a loyal friend, a splendid soldier and citizen, and a devoted husband, he had a happy faculty of making friends with his associates and others. His opinions were respected even by those who differed with him, and as a result he had a host of friends. As an engineer he never used "snap judgment." He collected available facts, and his decisions were always based upon and supported by these facts to which he applied a well-trained and methodical mind. A sterling character prevented him from rendering a knowingly questionable opinion or performing work not in keeping with the highest ethical standards.

He was a member of the Church Street Methodist Church of Knoxville; Knoxville Chapter, Sons of the Revolution; and the following Masonic Orders:

Masters Lodge No. 244, Free and Accepted Masons, of Knoxville; Tyrian Chapter No. 219, Royal Arch Masons, Stapleton, N. Y.; Empire Commandery No. 66, Knights Templar, Stapleton; Tennessee Consistory No. 1, Memphis, Tenn.; and Mecca Shrine of New York City.

In 1931 he was married to Emma Louise Jeffries of Devonshire, England, at Los Angeles, Calif., who survives him. They had no children. In addition to Mrs. Murphy, he is survived by a brother, Alonzo E. Murphy, and a sister, Edith Murphy.

Mr. Murphy was elected an Associate Member of the American Society of Civil Engineers on July 9, 1912.

MONT CAGLEY NOBLE, Assoc. M. Am. Soc. C. E.[1]

Died August 10, 1939

Mont Cagley Noble and his seventeen-year-old son, Robert, met a tragic death by drowning while on a vacation trip through the Minnesota-Ontario chain of lakes. Both were buried in Lincoln, Nebr., on August 18, 1939.

Mr. Noble was born in Nashua, Iowa, on March 9, 1894, the son of Henry and Mary (Cagley) Noble. Both parents were descendants of the early settlers of Nashua. After attending the elementary and high schools of that city, he entered Iowa State College at Ames, Iowa, in 1912, and was graduated in 1916, with the degree of Bachelor of Science in Civil Engineering. At intervals during his college course, Mr. Noble was employed as assistant county engineer of Chickasaw County, Iowa, on surveys, plans, and construction projects, and shortly after graduation from Iowa State he became assistant county engineer of Dickinson County, Iowa. He held this position until September, 1917, when he entered the Second Reserve Officers' Training Corps at Fort Snelling, Minn. Commissioned as a Second Lieutenant, Coast Artillery Corps (C.A.C.), he went overseas in December, 1917, with the 44th C.A.C. Regular Army. As First Lieutenant he acted as Batallion Reconnaissance Officer and Commanded Battery E, on the Argonne front when the Armistice was signed.

After resigning from the service in March, 1919, Mr. Noble entered the employ of the Nebraska State Department of Public Works and Buildings, serving as project engineer, district engineer, and from March, 1923, to January, 1927, as chief engineer in charge of the Bureau of Roads and Bridges.

In January, 1927, he became associated with the Armco Culvert Manufacturers Association as district engineer and regional manager in twelve Mississippi Valley states. In April, 1933, he entered the employ of the W. Q. O'Neill Company of Illinois (a subsidiary of the American Rolling Mill Company) as General Manager, and was located in Springfield, Ill. He held this position at the time of his death.

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[1] Memoir prepared by C. M. Hathaway, M. Am. Soc. C. E.

Mr. Noble was married to Jeanette Forke of Lincoln in 1921. Mrs. Noble and two sons, Dale and Richard, survive him.

He was an active member of the Central Illinois Section of the Society, having served on several committees. He was also a member of the Springfield Engineers Club, the Illinois Society of Engineers, the National Society of Professional Engineers, the American Legion, and the Chamber of Commerce of Springfield.

"Monty" Noble may be classified as a leader in his chosen calling, and he may be characterized as a capable engineer, a successful executive, a well-respected citizen, a man interested and active in his professional and civic societies, and a true friend and gentleman at all times.

Mr. Noble was elected an Associate Member of the American Society of Civil Engineers on August 30, 1926, and a Member on February 14, 1938.

EDWARD MENZEL NOON, Assoc. M. Am. Soc. C. E.[1]

DIED JUNE 12, 1940

Edward Menzel Noon was born at Nogales, Ariz., on March 15, 1901. He was the second of three sons of Adolphus Samuel Noon and Anna (Menzel) Noon. His father was one of the early self-made engineers in southern Arizona practicing mechanical and civil engineering, and contracting. Through his father's influence the son naturally turned to engineering as a profession.

Mr. Noon was graduated from the Nogales High School and then attended the University of Arizona, at Tucson, Ariz., for two years. He was elected a member of the Kappa Sigma fraternity and served as house manager of the Tucson chapter for a year. Later he transferred to the University of Illinois at Urbana, Ill., and received the degree of Bachelor of Science in Civil Engineering in 1926.

His first experience in engineering came in 1922 when he was employed by the Arizona State Highway Department as a rodman and chainman on construction survey and inspection of reinforced concrete culverts. In 1923 he became chief of party, with the Arizona State Highway Department, on location and construction surveys for hard-surfaced roads and connecting structures.

From June to September, 1924, he was resident engineer for J. B. Robinson, a contractor in Hermosillo, Sonora, Mexico. He was in charge of the construction of the sanitary sewerage system and a septic tank, as well as the design and construction of a water reservoir roof, and a pumping plant.

From September, 1924, to February, 1925, he was a resident engineer of Gila County, in Arizona, under James Parker, county engineer, in charge of final location, design, and construction of a heavy mountain road (Fossil Creek). In February, 1925, he was employed by the City of Los Angeles, Calif., as surveyor on construction surveys for streets, pavements, and sewers, and the location of a 1,500-ft viaduct. In May, 1925, he returned to Gila County as resident engineer on the same mountain highway that he had left in February due to snow. He remained here until September, 1925, when he left to attend the University of Illinois. While still attending the University of Illinois he did drafting and designing work, for the Leavitt Manufacturing Company, on portable wood and steel bleachers used in Soldier Field Stadium, Chicago, Ill.

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[1] Memoir prepared by M. L. Noon, Nogales, Ariz.

Mr. Noon was irrigation engineer from July, 1926, to May, 1927, with the Richardson Construction Company in the Yaqui Valley, Sonora, Mexico, in charge of engineering on an operating 100,000-acre irrigation project, and town-site locations. He then returned to Los Angeles and remained there until July, 1929. He was first with the Southern California Edison Company for three months, then with the Los Angeles County Sanitary Division, and finally with the Streets Department on special work on recreational areas and street widening.

In July, 1929, he was employed by the Standard Fruit and Steamship Company in La Ceiba, Honduras, Central America. Mr. Noon went to Honduras as a draftsman and designer. In December, 1929, he was made principal assistant engineer in charge of topographic work, and location and construction of canals. In April, 1930, he became engineer in charge of all work and construction of two 23,000-gal-per-min pumping plants. Under his supervision at this time were about five hundred laborers and twenty engineers. Mr. Noon remained in this capacity until October, 1930, when he was forced to leave on account of his health.

He went to New York, N. Y., and in January, 1931, he joined the forces of C. O. Tappan. While with Mr. Tappan he worked on the design of sewerage systems and disposal plants for small communities, and the preparation of plans and specifications. In addition, he investigated and made preliminary designs of hydraulic and dock projects.

Due to reduced construction work in New York City he left the services of Mr. Tappan in March, 1933, and went with the U. S. Coast and Geodetic Survey. There he compiled data and did drafting work on aerial surveys. The course of this work carried him down the Atlantic coast and in August, 1933, he resigned and was employed by Gregory and Bernard Engineers, at Crowley, La., and Orange, Tex. In Crowley he engineered irrigation projects in the rice fields, and in Orange he was in direct charge of engineering and design of irrigation system of canals and laterals, the installation of four 23,000-gal-per-min turbine pumps, and pump buildings. Following the completion of the Orange project he became city engineer of Jennings, La., planning and engineering city improvements including sewerage, water systems, and paving.

In July, 1937, he accepted a position with the Martin Engineering Company of Maracaibo, Venezuela, as superintendent of construction. Mr. Noon constructed hundreds of concrete block houses for native laborers of the Gulf Oil Company and the Standard Oil Company in Cabimas, and Lugunillas, Venezuela. At the time of his death he was supervising placement of 55-ft sheet piles and river-bank protection preliminary to the construction of docks on the Catumba River, at Encontrados, Venezuela.

Mr. Noon met his death at the hands of a supposedly trusted watchman in his employ who apparently went berserk with a machete. His death and murder came as a shock to all who knew him, especially the Venezuelans he worked with, as he was quiet, understanding, and well liked by every one. He is buried at Encontrados, but his remains will be removed to Nogales at a later date as permitted by Venezuelan law.

Mr. Noon was elected an Associate Member of the American Society of Civil Engineers on February 24, 1931.

ALOIS PHILLIP POIROT, Assoc. M. Am. Soc. C. E.[1]

Died November 9, 1939

Alois Phillip Poirot was born in Belleville, Ill., on March 5, 1883, the son of Severin Poirot and Susan (Gundlach) Poirot, prominent pioneer residents of that city.

Mr. Poirot attended the Catholic elementary schools at Belleville and was graduated from the Belleville Township High School in 1902. In 1906 he was graduated from the University of Illinois, at Urbana, Ill., with the degree of Bachelor of Science in Civil Engineering. He continued his technical education at the university with a postgraduate course in reinforced-concrete construction, part of which consisted of the preparation of a thesis on the design and construction of the Butler Brothers Building at St. Louis, Mo.

Entering the services of the Wabash Railway Company in 1906, he received his first practical experience in bridge design. In 1907 he entered the services of the Missouri Pacific Railroad Company as engineer in charge of field party on location work.

In 1908 he formed a partnership with the late Joseph M. Slater,[2] M. Am. Soc. C. E., of St. Louis, under the firm name of Slater and Poirot. The firm was engaged in engineering and construction work, and Mr. Poirot's duties consisted principally of designing and superintending the construction of reinforced-concrete bridges and buildings, one of the outstanding structures being the James River Bridge.[3] This was a reinforced-concrete bridge across the James River at Richmond, Va., and had been submitted in competition with the design furnished by the City of Richmond. While connected with this firm, Mr. Poirot also designed the Commercial Building at Belleville. This was a five-story office and store building and was one of the first reinforced-concrete buildings to be constructed in Belleville.

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[1] Memoir prepared by F. R. Friedewald, Assoc. M. Am. Soc. C. E.

[2] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 100 (1935), p. 1733.

[3] *Loc. cit.*, Vol. 95 (1931), p. 244.

From 1912 until 1916 Mr. Poirot was engaged in the private practice of engineering and also was employed as chief engineer of the Belleville and Interurban Railway System, the Belleville-Mascoutah Traction Company, and the St. Louis and St. Libory Railway Company.

From 1916 until 1925 he was engaged in private engineering and construction work, and was particularly interested in the field of competitive design and construction of bridges in south-central Illinois, one of his outstanding structures being the New Athens, Ill., Viaduct, which was constructed in 1917 and 1918. This structure was of the T-beam type and utilized the split column pier, which was for the purpose of taking care of the necessary expansion and contraction of the structure.

Mr. Poirot constructed the Sac River Bridge near Ash Grove, Mo., in 1926. It consisted of three 100-ft arch spans. From 1926 to 1936 he was engaged principally in the design and construction of water works distribution and filtration systems, some of which were located at Chester, Fairfield, New Athens, Waterloo, Sparta, and Carbondale, Ill. Mr. Poirot was considered an authority on this work and was frequently called upon to serve in a consulting capacity in southern Illinois and Missouri. Because of ill health, Mr. Poirot had to retire from active participation in this work in 1936, and it was at this time that he became particularly interested in rice farming property which he had acquired near Roe, Ark.

Mr. Poirot's undertakings were outstanding, in that most of the work in which he was engaged was of his own design. He showed unusual ability and used his ingenuity and originality to the advantage of all concerned. In addition to being particularly interested in the engineering and construction field, he also took a great interest in his own community. He was a member of the Order of Elks and the Knights of Columbus, and was a director of the St. Clair National Bank of Belleville. He also was a member of the American Rice Growers Association.

On June 8, 1922, he was married to Anne Ludwina Voigt of Horton, Kans. He is survived by his widow; two brothers, Severin A. and Eugene M.; and four sisters, Elenora Poirot, Malinda Hambuechen, Susan Frein, and Rose Jantzen.

Mr. Poirot was elected an Associate Member of the American Society of Civil Engineers on August 31, 1915.

JOHN MARBLE RACE, Assoc. M. Am. Soc. C. E.[1]

DIED NOVEMBER 27, 1940

John Marble Race was born on March 19, 1869, at Charlton, Mass., the son of Harry and Melissa (Brown) Race. Mr. Race obtained his early education in private schools in Chicago, Ill., and in the high school at Great Barrington, Mass. His later scholastic and engineering education was obtained by hard study, self-cultivation, and experience.

From 1892 until 1898, Mr. Race served as rodman, draftsman, and transitman with J. P. and W. H. Barnes, and with T. Z. Potter, civil engineers of Berkshire County, Massachusetts. From 1898 to January, 1901, he served as a draftsman and later as assistant engineer with the late Charles W. Leavitt,[2] M. Am. Soc. C. E., civil and landscape engineer of New York, N. Y. The following two years, 1901 to May, 1902, he served with the U. S. War Department, in the provisional government of Cuba at Santiago De Cuba, where he held the rank of assistant engineer on water and sewer work. Later he acted as supervising engineer on construction of asphalt paving for the city. For six months, until the end of the American occupation of Cuba, he was resident engineer in charge of sewer work, and supervised the construction of twelve miles of sewers.

From the end of the American occupation in May, 1902, until 1906, he was associated with Barnes and Farnham, Civil Engineers, of Pittsfield, Mass., as assistant engineer, and was employed on various municipal works and surveys. He also made topographical surveys for the Century Map Company of New York City.

In 1906 he returned to Cuba as an assistant engineer with Hobby and Binckley, Consulting Engineers, of Habana, and was engaged on railway surveys for a year or more. From then until late in 1909 he served as a first assistant engineer in the Public Works Department on surveys for the drainage of the Roque Valley, Matanzas Province, and later as an assistant engineer on sewer and water projects for the City of Habana.

Upon his return to the United States, Mr. Race was for a while in private practice and later, until 1911, was again employed as assistant engineer with Mr. Leavitt. He was in charge of preliminary estimates on an $18,000,000 project for the rehabilitation of Brigantine Beach in New Jersey.

From 1911 to 1913, he was principal assistant engineer with Barnes and Spaulding, Civil Engineers, of Pittsfield, and later from 1913 to 1917 was associated with Mr. Barnes under the firm name of Barnes and Race, Civil Engineers, at Pittsfield. In 1918 he was elected town engineer and selectman of Great Barrington, and was re-elected in 1919. He resigned, however, to go to Santo Domingo, Dominican Republic, as resident engineer in the Public

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[1] Memoir prepared by H. E. Hyde, M. Am. Soc. C. E.

[2] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1865.

Works Department, where he was in charge of all public works in the Province of Azua, including port and dock work and highways. He later handled irrigation work for the Barahona Sugar Company in the same province.

In 1921, due to poor health, he returned to the United States and engaged in private practice for a while at Great Barrington. From 1922 until 1930, Mr. Race served as assistant engineer for various consulting engineers in Springfield and Pittsfield, Mass., being engaged on municipal projects, topographical surveys, etc. He was assistant engineer with A. A. Fobes, civil engineer, of Pittsfield for nearly three years, engaged in general practice; for three years with A. J. Kohlofer, civil engineer, of Pittsfield; and for two years with Durkee, White and Towne, Civil Engineers, of Springfield, engaged on surveys, road construction, drainage, and water supply. He also supervised the installation of eight miles of water mains in the town of Southwick, Mass.

From 1930 to 1934 Mr. Race was in business for himself, engaged on topographical surveys and on plans and surveys for the Bowles (Agawam) Airport. He also did survey and subdivision work for the Hamilton Lakelands Development at Holland, Mass. From 1934 until the summer of 1937, he was with the Public Works Administration (PWA) as an engineer inspector and was engaged on two large street projects, totaling over a million dollars, in Worcester, Mass., and on sewer projects at Deerfield, Mass. In the fall of 1937 he was for a time with the Works Progress Administration (WPA) of Massachusetts as an engineer inspector on work at Springfield and in the surrounding towns. In 1938 he returned to the PWA as engineer inspector in charge of work at the Quabbin Reservoir, Boston Water Supply. He served in this capacity until late in 1939, when on account of illness he returned to his home at Agawam, Mass., where he died on November 27, 1940.

Mr. Race was an accomplished draftsman and was considered the best in Berkshire County. He apparently inherited his artistic talent, as two of his near relatives were Leonard Volk, a noted sculptor, and Douglas Volk, an internationally known portrait painter. Mr. Race was very fond of drawing and painting.

He was a man of marked stability of character, and was painstaking and thorough in all of his work. Those who employed him thought highly of him. He was conscientious and honest in all of his work and dealings, a hard worker, and a man of diversified experience. His good nature, even temper, and careful manner of expressing himself were manifest in all of his undertakings and earned for him the liking and good will of those with whom he came in contact. He was a man one instinctively liked and trusted.

He was married on October 31, 1900, to Bertha Nicolai and is survived by his widow and two brothers, Charles A. and William B. Race.

Mr. Race was elected an Associate Member of the American Society of Civil Engineers on March 5, 1902. He became a life member on January 1, 1937.

THOMAS HERBERT RITNER, Assoc. M. Am. Soc. C. E.[1]

Died September 10, 1939

Thomas Herbert Ritner was born on June 3, 1897, in Philadelphia, Pa., the son of Walter C. and Annie (Merton) Ritner. His forebears were among the earliest settlers in this country, and his great-grandfather, Joseph Ritner, was Governor of Pennsylvania in 1835.

Thomas Ritner was educated in the public schools of New York, N. Y., and was graduated from The Pennsylvania State College, at State College, Pa., in 1922, with the degree of Bachelor of Science in Civil Engineering. He was popular as a student, a member of Phi Gamma Delta and several honorary societies, and entered into athletic activities, receiving letters in football, baseball, and basketball. His education was interrupted when he enlisted in the 59th Company Transportation Corps on May 30, 1918. He served one year in France and resumed his studies in September, 1919, following his discharge from the service early that same month.

It was only natural upon completion of his engineering education at "Penn State" that Mr. Ritner should turn to heavy construction. His father had been superintendent of construction on many large bridge projects, and he had spent several summer vacations working on these projects. From June, 1922, to June, 1923, he acted as assistant superintendent in charge of replacements and additions to the Marion Power Station for the Public Service Production Company at Newark, N. J. Upon completion of that job he served one year in the contract department of the Westinghouse Electric and Manufacturing Company in East Pittsburgh, Pa. This was the only non-engineering engagement in Mr. Ritner's career.

From May, 1924, to November, 1925, Mr. Ritner served as assistant to the chief engineer and superintendent of construction for the C. F. McAvoy Company, in Newark, on construction of dams for the North Jersey District Water Supply Commission at Wanaque, N. J. In December, 1925, he joined the Parker and Graham organization and served as engineer in charge of construction of the Erie Railroad elevation at Paterson, N. J., and bridges at Paterson, Franklin, Glen Rock and New Brunswick, N. J. Upon completion of this work in October, 1928, he served in the same capacity with the A. H. Earle Company on construction of the Jones Beach Causeway.

From August, 1929, until his death ten years later, Mr. Ritner served as engineer, estimator, and superintendent of construction on many varied projects in the east, extending from New York to Florida, such as docks, bridges, retaining walls, and construction of the foundations of a number of the largest buildings and structures at the New York World's Fair. During that time he was associated with Sexton, Inc., Snyder Engineering Company, P. T. Cox Construction Company, Young Fehlhaber Pile Company, Slattery Contracting Company, and the William P. McDonald Construction Company.

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[1] Memoir prepared by Ellis E. Paul, M. Am. Soc. C. E.

On September 19, 1928, Mr. Ritner was married to Dorothy Durbin of Narberth, Pa., who survives him. He is also survived by two sons, Thomas Herbert, Jr., and Joseph Walter, and by his daughter, Beverlyn Stewart.

He was a member of the Munn Avenue Presbyterian Church of East Orange, N. J.

Mr. Ritner was elected a Junior of the American Society of Civil Engineers on May 28, 1923, and an Associate Member on July 15, 1929.

GRIER RALSTON SMILEY, Assoc. M. Am. Soc. C. E.[1]

DIED JUNE 10, 1941

Grier Ralston Smiley was born on a farm near Moffatts Creek, Va., on February 6, 1880, the son of Thomas M. and Ellen (Lantz) Smiley. He was educated at Washington and Lee University, in Lexington, Va., and was graduated in June, 1902, with the degrees of Bachelor of Arts, and Bachelor of Science in Civil Engineering. In 1934 he was awarded a Phi Beta Kappa key at the University for outstanding engineering achievement.

After some three months in the service of the Virginia Iron, Coal and Coke Company, he entered the service of the New York Central Railroad Company, serving as chainman, rodman, instrumentman, draftsman, and track supervisor until November, 1905, when he entered the service of the Florida East Coast Railway Company as assistant resident engineer on construction of the extension from Miami to Key West, Fla. He was promoted to resident engineer in September, 1906, and to division engineer in March, 1909.

Mr. Smiley was largely responsible for the design and construction of the extensive concrete bridge structures which, until the storm of September 2, 1935, carried the Florida East Coast Railway to Key West. After the 1935 storm the railroad from Miami to Key West was abandoned, and the bridge structures were remodeled under the direction of B. M. Duncan to support the Miami to Key West State Highway.

On May 28, 1912, the Florida East Coast Railway extension having been completed, Mr. Smiley entered the service of the Louisville and Nashville Railroad Company as resident engineer in charge of some heavy construction on the new line between Winchester and Irvine, Ky. On July 15, 1915, he was assigned to represent the interest of the Nashville, Chattanooga and St. Louis Railway Company on construction of a bridge over the Ohio River at Metropolis, Ill., returning to the Louisville and Nashville Railroad Company on January 15, 1917, as special engineer on surveys for reconstruction of bridges over Chef Menteur and Rigolets Pass, Louisiana.

On September 1, 1917, he was placed in charge of the Miscellaneous Department of the chief engineer's office of the Louisville and Nashville Railroad Company. On October 1, 1920, he was made chief engineer of construction;

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[1] Memoir prepared by C. H. Blackman, M. Am. Soc. C. E.

on April 1, 1931, assistant chief engineer; and on August 1, 1933, chief engineer, which position he held until his death.

During his service with the Louisville and Nashville Railroad Company, he had immediate charge of a large amount of second-track construction in the mountainous section of eastern Kentucky, constructing a line through the mountains from Harlan, Ky., to Hagan, Va.; construction of a bridge over the Ohio River at Henderson, Ky.; and much other important work.

It was while engaged on the Florida East Coast Railway work that Grier Smiley met Miss Nora J. Krome of Edwardsville, Ill., a sister of the late William J. Krome, M. Am. Soc. C. E., the chief engineer of construction. They were married in 1911. He is survived by his widow and one daughter, Helen (Mrs. William Bowman Cutter); a brother, William V. Smiley; and a sister, Caroline (Mrs. Richard Hogshead).

Grier Smiley was a man of lovable character, who endeared himself to all who came in contact with him. He was a cultured Virginia gentleman, and when asked during his illness where he would want to be buried he answered by humming a line from "Carry Me Back to Old Virginny." That tune was played while he was being placed in his grave in the churchyard behind the Moffatts Creek Presbyterian Church, where he had been the organist in his boyhood days.

He served as the first President of the Kentucky Section of the Society. He was a Director of the American Railway Engineering Association and took an important part in the proceedings of that Association. He had also served as President and Director of the Engineers and Architects Club of Louisville, Ky. Since 1932 Mr. Smiley had been a member of the Advisory Committee of business executives of Louisville for the Speed Scientific School, University of Louisville. He was a member of the Presbyterian Church, Pendennis and Filson clubs, and Big Spring Golf Club of Louisville. His passing at the early age of sixty one is an irreparable loss to the railroad industry and to the engineering profession.

Mr. Smiley was elected an Associate Member of the American Society of Civil Engineers on April 1, 1908.

WILLIAM FREDERIC STARKS, Assoc. M. Soc. C. E.[1]

Died November 1, 1939

William Frederic Starks, who died as a result of an automobile accident on November 1, 1939, was born in Newburgh, N. Y., on January 12, 1875. His parents were Charles J. and Elizabeth M. (Greene) Starks.

His education was received in the public and private schools of Newburgh. He obtained his engineering training with local engineers, and with the engineering departments of the New York Central Railroad Company and the Central Railroad Company of New Jersey.

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[1] Memoir prepared by B. H. Wait, M. Am. Soc. C. E.

After serving his apprenticeship with the railroad he associated himself with William H. Bowne, Assoc. M. Am. Soc. C. E., of Glen Cove, N. Y., in general engineering and surveying. Their company took an active part in the development of Nassau County, New York.

In 1911 Mr. Starks was appointed superintendent of highways for Nassau County and later, in 1922, was appointed county engineer, serving until 1936 when he retired on account of ill health. In these positions he had charge of practically all highway and bridge work, and much sewer and miscellaneous work in the development of the county from a farming district to a highly suburban territory. From 1936 until his death he was in private and consulting engineering work.

Mr. Starks was a former president of the New York State Association of County Superintendents of Highways, a Past-Master of the Glen Cove Masonic Lodge, and a member of the Nassau-Suffolk Engineering Society.

Mr. Starks was well known throughout the county and state as a man of integrity and outstanding engineering ability. Much of the work which he did in the development of Nassau County was made available to, and was of great help to, other engineers in the state in the development of other county systems.

He was married to Ida Baldwin on October 1, 1908, who survives him.

Mr. Starks was elected an Associate Member of the American Society of Civil Engineers on November 26, 1918.

JOSEPH CONRAD STEGNER, Assoc. M. Am. Soc. C. E.[1]

DIED DECEMBER 29, 1939

Joseph Conrad Stegner was born in Fulda, Minn., on January 5, 1883, the son of Joseph Andrew and Matilde Greenfield Stegner. At an early age he went with his parents to Washington Territory, and most of his youth was spent in the eastern portion of the Spokane Valley. He was graduated from what was later Lewis and Clark High School in Spokane, and then began work in the engineering department of the Northern Pacific Railway Company, where his advancement was rapid.

Included in the work that he did for the railroad was service in the capacity of resident engineer during construction of the Western Dakota Railway, a branch line in North Dakota. He also did considerable work on the design of the company's grade separation through the City of Spokane and rebuilding yards in preparation for the separation.

In 1915 Mr. Stegner entered the service of Pend Oreille County under County Engineer H. A. Sewell, M. Am. Soc. C. E., becoming county engineer himself in 1919 and 1920. Retiring to private life, he was for two years with the Diamond Match Company operations on Tacoma Creek. In 1923 he returned to the position of county engineer, which he held for the remainder of his life. At the time of his death only one other county engineer in the state held a record of longer service in that capacity. During the years that he held this position, there were many changes in the methods of handling road funds, as well as in construction practices. Mr. Stegner had much to do with the development of the permanent highway, the farm-to-market road, and succeeding plans for conducting the county's program of road development. He was an important factor in the progress along these lines.

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[1] Memoir prepared by H. A. Sewell, M. Am. Soc. C. E.

In 1932 he was president of the State Association of County Engineers. During the last two years of his life, he was a member of the Board of Directors of the Newport Consolidated School District.

On August 9, 1905, he was married to Welthy Grace Wills. She died on November 15, 1940. He is survived by two children, Robert Stegner and Matilde (Mrs. Harold) Buchholz.

Mr. Stegner was elected an Associate Member of the American Society of Civil Engineers on June 1, 1925.

SAMUEL STRUMER, Assoc. M. Am. Soc. C. E.[1]

Died December 7, 1940

Samuel Strumer was born in New York, N. Y., on May 17, 1894, the eldest son of Joseph and Sadie (Schindler) Strumer. He received his early education in the grade and high schools of New York City. In 1917 he was graduated from Cornell University, in Ithaca, N. Y., with the degree of Civil Engineer. He studied architecture at Columbia University, in New York City; art at the Sorbonne (University of Paris) and at the Julien Academie; and French at the Alliance Français in Paris, France.

In 1914, 1915, and during the summer of 1916, he was engaged by the Public Service Commission, 1st District, in New York City as a junior assistant engineer on subway construction in connection with the 7th Avenue Subway and the East River tunnels from Manhattan to Brooklyn.

In 1917, one month after the United States entered the World War, he volunteered and was commissioned a Second Lieutenant in the Coast Artillery Corps (C.A.C.). He was soon promoted to a First Lieutenant and went to France with the 48th Artillery, C.A.C. After his discharge from active duty in 1919, he accepted a commission as First Lieutenant in the C.A.C. Reserves. In 1924 he was commissioned a Captain in the C.A.C. Reserves and in 1930 he received his commission as Major. An early and an intensely serious student of anti-aircraft problems, he soon became expert in this field and served as an instructor of anti-aircraft artillery from 1925 to 1933. This instruction was given to reserve officers in Troop School conducted by Headquarters, C.A.C. Reserves, Second Military Area. His thorough engineering training coupled with his knowledge of anti-aircraft tactics made him an invaluable officer. In 1933, Major Strumer was one of the few reserve officers chosen to participate in the joint operations of the Air Corps and Anti-Aircraft Artillery in studying the defense of Fort Knox, Ky. While there, he served as Assistant Intelligence Officer. Upon the findings and recommendations of this commission, Fort Knox was later chosen as the national storehouse for the gold reserve of the United States. Major Strumer went into active training almost every alternate summer. His last command was with the 910th C.A.C.—Anti-Aircraft.

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[1] Memoir prepared by Mrs. Samuel Strumer of White Plains, N. Y., and Julius Strumer of New York, N. Y.

From 1919 to 1921, Lieutenant Strumer was in private engineering practice in New York City, being employed by the Fougner Concrete Steel Company as a structural draftsman on the design of reinforced-concrete buildings.

From 1922 to 1923 he was employed by the Transit Commission of New York City as a structural steel draftsman on the design and drafting of subway and elevated structures such as the Queens Plaza Extension and 42d Street crosstown subway in New York City. He was also engaged by the Board of Education of New York City as a structural draftsman, and designed foundations and superstructures of various school buildings, such as the Girls' Commercial High School and the James Madison High School, both in Brooklyn.

From 1923 to 1926, Captain Strumer was employed by the Department of Public Works, Division of Sewers and Highways, Borough of Queens, as a draftsman on the design of sewage disposal plants, sewer systems, highway bridges, bulkheads, buildings, boardwalks, and other public improvements. During 1926 he was also employed by the Division of Design, Board of Transportation, New York City, as a draftsman engaged in the design of subway structures of steel and reinforced concrete. During this period he designed the 8th Avenue Station at 23d Street, New York City.

From 1926 until his untimely death, Major Strumer was in the employ of the City of New York, Department of Buildings, later known as the Department of Housing and Building. As an engineer he checked applications and plans for the erection of every conceivable type of building and assumed full responsibility for the safety of these structures. Notable among these is the vast housing project known as "Parkchester," which consists of fifty-one individual elevator apartment buildings covering 35½ acres, and which is the largest housing project in the world. At the time of his death, he was chief plan examiner of the Bronx; he had also served as chief engineer of the Bronx. Besides his regular daytime duties, he was instructor of Plan Reading and Estimating at the Bushwick Evening Trade School, in Brooklyn, from 1923 to 1933.

In addition to his professional practice, Major Strumer was vitally interested in community life. He was a leader of the Fenimore Cooper Council, Boy Scouts of America, and chairman of Troop No. 22, in White Plains, N. Y. He was the forceful organizer of the White Plains Post of the Jewish War Veterans, U. S. Army, and was elected the first commander of that post. He was senior vice-commander of Westchester County-Jewish War Veterans, U. S. Army; secretary and member of the Board of Trustees of Temple Israel, White Plains; member of the Board of Governors of the Men's Club, Jewish Community Center of White Plains; and secretary of the Central Veterans' Committee, White Plains. He was a charter member of the Coast Artillery Association, U. S. Army, and a member of the following organizations: Reserve Officers of the U. S. Army; Army Ordnance Association; Officers' Club of the Army and Navy; Society of the American Military Engineers; Westchester County Chapter, Reserve Officers Association, U. S. Army; Cornell Society of Engineers; New York State Society of Professional Engineers; National Society of Professional Engineers; French Universities Post, American Legion; Amex Post, Veterans of Foreign Wars; and Jupiter Lodge, Independent Order of Odd Fellows.

Measured in years, his life was a short one—46 years; measured in terms of accomplishment, he lived a long life, a useful life, a full life. He was a human dynamo of activity; an idealist; a perfectionist; a man whose ideals of honesty, conscientiousness, and integrity won for him the high regard of his supervisors, the sincere and deep respect of all who knew him, and a multitude of real friends.

Major Strumer was a loving and beloved husband; an interested and devoted father. He is survived by his widow, Mildred Reiter Strumer, to whom he was married on June 4, 1922; a daughter, Naomi Louise; a son, Josef Newton; two sisters, Rose and Frances; and three brothers, Saul, Julius, and William.

Major Strumer was elected an Associate Member of the American Society of Civil Engineers on September 12, 1938.

OWEN ADELBERT WAIT, Assoc. M. Am. Soc. C. E.[1]

DIED JANUARY 10, 1940

Owen Adelbert Wait was born at Groton, N. Y., on July 31, 1874, the son of Seneca Adelbert Wait and Ida May (Boswick) Wait (born Pamela E. Conrad).

Mr. Wait was graduated from the high school at Groton as valedictorian. In 1898 he was graduated from Cornell University, at Ithaca, N. Y., with the degree of Civil Engineer.

His early training in the engineering field was with the Groton Bridge and Manufacturing Company where he started in September, 1892, as blueprint and office boy. He held this job for seven months, and then was promoted to the drafting force. He worked as draftsman with this company until September, 1894, during summer vacations from 1895 to 1897, and subsequent to his graduation from Cornell University from June to November, 1898. Mr.

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[1] Memoir prepared by Walter J. Wood, Assoc. M. Am. Soc. C. E., with the assistance of O. M. Stultz, Esq., Los Angeles, Calif.

Wait then was employed by the Union Bridge Company until February, 1900, on detailing bridge and structural steel work.

From February, 1900, to April, 1905, he was with the Niagara Falls Power Company where his work was varied. He worked on designs, details, and writing specifications for structural steel work; on shop and field inspection of structural steel work; on studies and estimates; and during 1903 and 1904 had charge, under the late A. H. Van Cleve,[2] assistant engineer, and the late W. A. Brackenridge,[3] resident engineer, Members, Am. Soc. C. E., of the designing and detailing of all steel work designed in the office of Niagara Falls Power Company for the Niagara Falls Power Company and Canadian Niagara Power Company.

Mr. Wait was designing engineer for the Great Northern Power Company at Duluth, Minn., from April, 1905, to January, 1907. During this engagement he was connected with the construction of a large hydroelectric plant. From February, 1907, to November, 1910, he was employed as assistant engineer by the Niagara Falls Water Works, at Niagara Falls, N. Y.

In November, 1910, he accepted a position with the Southern California Edison Company of Los Angeles, Calif., as assistant engineer, which position he held until July 1, 1923. From the latter date until January 1, 1930, he served as division head in the Engineering Department and thereafter occupied the position of chief of hydrographic division of the Southern California Edison Company, until his retirement on September 1, 1934.

Retirement proved to be an unsatisfactory sort of existence to a man whose life was built on hard work, and so Mr. Wait started looking for something to occupy his time. In October, 1935, he became connected with the Los Angeles County Flood Control District with whom he was employed until his death. His long experience on hydrographic work, together with his analytical ability, proved very valuable.

Mr. Wait was a member of the Masonic Lodge, being a Knight Templar for many years. His principal hobbies were studying, fishing, gardening, and building short-wave radios. During his later years, at least, he spent much of his spare time working at the office even on his days off. It was only as the result of insistence from others that he would take a vacation to which he was entitled. His capacity for work and his desire for precision and attention to details in his work were outstanding.

Mr. Wait was a very shy and unassuming man. He was greatly loved by those fortunate enough to know him well. Only a few persons were aware of his philanthropy because he never mentioned it nor would he admit it. His life was devoted to his mother until her death in 1935 and thereafter to his brother's family. He was generous to a fault.

He is survived by a brother, O. LeRoy Wait; two nephews, Owen Leon Wait and William Argyle Wait; and a cousin, Lyman Wait.

Mr. Wait was elected an Associate Member of the American Society of Civil Engineers on December 6, 1905, and became a Life Member on January 1, 1940.

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[2] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 100 (1935), p. 1742.

[3] For memoir, *loc. cit.*, Vol. 96 (1932), p. 1419.

RALPH DARWIN WICHMAN, Assoc. M. Am. Soc. C. E.[1]

Died September 22, 1940

Ralph Darwin Wichman was born in Sunderland, Mass., on January 12, 1883, the son of Flora (Clark) and Christian Frederick Wichman. He attended grammar school in Velasco, Tex., and Missoula, Mont. His rudi mentary technical education was obtained by two years' study in San Francisco, Calif., at the Humboldt Night School, one of the first in that city. This school specialized in mechanical drawing, physics, chemistry, mathematics to calculus, and other scientific courses. An International Correspondence School course in marine engineering, and private engineering tuition for a year, completed his formal education.

He started working as a draftsman with the Risdon Iron Works in San Francisco in 1905, holding this position for three and one-half years. Following short engagements with Henshaw Buckley Company, and the Southern Pacific Company on steam generating plant design, Mr. Wichman was with the Mt. Whitney Power and Electric Company at Visalia, Calif., for six years, from 1912 to 1918, as chief draftsman and construction engineer on hydroelectric plant design and construction, which included construction of substations and street-lighting systems.

For a period of three months in 1916, he was with the National Guard of California, at Nogales, Mexico, on camp drainage and road building. In 1918 he returned to San Francisco, where he was employed as designer on piping layouts for destroyers by the Bethlehem Shipbuilding Corporation, Ltd., for a period of six months. Following this he spent six months as designer on a refinery layout for the Shell Oil Company. In 1919, Mr. Wichman started an engagement, which lasted for a year and a half, as construction engineer and designer with the Burma Corporation, Ltd., at Namtu, Northern Shan States, Burma, India. This work involved the construction of a smelter and substation design.

After traveling for four months, he returned to the United States, where he held the position of designer on wood and concrete structures on canals with the Imperial Irrigation District at Calexico, Calif. After six months in that position, he returned to San Francisco, in 1922, where he joined the engineering forces of the Pacific Gas and Electric Company for a period of three and one-half years, as electrical engineer, in charge of design for transmission and distribution substations, steam plants, and the like.

After twenty years of experience and study, Mr. Wichman's engagements, frequently combining the civil, electrical, and mechanical engineering fields, disclosed his unique ability to coordinate theoretical analysis with practical application. It was this ability that made his services important and desirable also in his future work in the study and design of oil refinery plants, hydroelectric projects, boiler and heating plants, and sewage treatment works.

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[1] Memoir prepared by A. V. Bowhay, Assoc. M. Am. Soc. C. E., in collaboration with Marvin Anaya.

In 1925 he was offered a position as designer with the Shell Oil Company at San Francisco, where for about two years he was engaged in the layout and detail of a continuous converter plant for separating distillate from residue. Accepting an opportunity in 1928 as structural designer in the Division of Design, Bureau of Engineering, of the Department of Public Works of the City and County of San Francisco, Mr. Wichman quickly demonstrated the value of his exceptional and varied experience. In the Civil Service examinations of 1931, for mechanical engineering designers, he passed well up on the list and was appointed immediately. With the exception of a break, due to shortage of funds, for a year in 1932, he served in this position until his death. As mechanical designer, his services concerned the design, contract specifications, and estimates of boiler and heating plants, sewage treatment works, and similar projects, as well as preliminary reports and plans on many projects in the mechanical engineering field. His service as principal assistant to W. H. Ohmen, chief designer, on the Garbage Incineration Report of 1929, presented by the late Michael M. O'Shaughnessy,[2] M. Am. Soc. C. E., is an example of the latter.

Mr. Wichman was the usual studious type of engineer, and was energetic in his efforts to maintain high personal professional standards. He kept an extensive private library of reference books on all engineering subjects. Among his colleagues and coworkers, he was a ready source of special information, which he was ever genial and companionable in giving. Mr. Wichman was licensed under the laws of the State of California as a professional civil engineer under Certificate No. 2526. He was a resident member of the San Francisco Section of the Society.

He is survived by his three children, a son, Ralph Darwin Wichman, Jr., and two daughters, Dorothy (Mrs. J. T. Archibold), and Zelda (Mrs. D. E. Knowland). To them he was a good companion and a devoted father. He is also survived and held in loving memory by his mother; his two sisters, Mrs. Minor R. Wallace and Mrs. Christina L. Wegman; and a brother, Neil Wichman.

Mr. Wichman was elected an Associate Member of the American Society of Civil Engineers on March 12, 1923.

ROBERT GOODLIVE BAINTER, Jun. Am. Soc. C. E.[1]

DIED JANUARY 1, 1941

Robert Goodlive Bainter, the son of Ernest C. Bainter and Mabel (Goodlive) Bainter was born in Zanesville, Ohio, on March 19, 1914. He grew up in Zanesville, attending both public school and high school there. Following his graduation from Zanesville High School he entered Ohio State University, at Columbus, Ohio, in the fall of 1931, but left in 1934 to take a position with the Soils Erosion Service, U. S. Department of the Interior, on a flood-control project with headquarters in Zanesville. He was engaged in surveying, timber cruising, stream gaging, hydrology, and soil studies.

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[2] For memoir, see *Transactions*, Am. Soc. C. E., Vol. 100 (1935), p. 1710.

[1] Memoir prepared by a Committee of the Central Ohio Section consisting of C. H. Shepard, Jun. Am. Soc. C. E., and R. R. Litehiser, M. Am. Soc. C. E.

After a year with the Soils Erosion Service, Mr. Bainter joined the U. S. Engineer Department at Zanesville as rodman. He continued in this capacity, acquiring experience in soil mechanics and earthwork construction, until June, 1938, when he was employed as an assistant in the Ohio State Highway Testing Laboratory at Columbus. That fall he returned to Ohio State University, working part time at the Highway Testing Laboratory. He completed his engineering course at the University and was graduated with the degree of Bachelor of Science in Civil Engineering in 1939. Upon graduation, Mr. Bainter resumed full-time work in the Highway Testing Laboratory in soil testing and the application of soil mechanics to highway construction, in which he was engaged at the time of his death.

Robert Bainter was a young man of the highest character and an engineer of much promise. He was well prepared for work in a new field of engineering, and his untimely passing is a distinct loss. As a youth, Robert displayed the traits that later marked him as a man. He was active as a Boy Scout, and at the age of fourteen was a confirmed member of St. John's Lutheran Church. He was a leader in Sunday School and young folk's work in the Church. Later, at Ohio State, he was active in the Reserve Officers' Training Corps and was a member of Pershing Rifles. He was guided in all his endeavors by the motto, "If it is worth doing at all, it is worth doing well."

He was married on Christmas Day in 1935 to Margaret Louise Pennell, who survives him, together with their two children, James Robert and Marianne.

Mr. Bainter was elected a Junior of the American Society of Civil Engineers on October 16, 1939.

HUGH B. GIBSON DUNLOP, Jun. Am. Soc. C. E.[1]

Died October 10, 1940

Hugh B. Gibson Dunlop was born in Jamestown, N. Dak., on May 4, 1910, the son of B. Gibson Dunlop and Grace Dunlop.

He was a member of the graduating class of June, 1936, at the State College of Washington at Pullman, Wash., receiving the degree of Bachelor of Science in Civil Engineering.

Immediately after leaving college he was employed by the United States Government as rodman with the U. S. Geological Survey during June and July, 1936.

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[1] Memoir prepared by Fred J. Sharkey, Assoc. M. Am. Soc. C. E.

He resigned from the U. S. Geological Survey upon receiving an engineering appointment with the U. S. Bureau of Reclamation as rodman on topographic surveys pertaining to construction of the Columbia Basin Project. From July, 1936, until his untimely death he was continuously employed by the Bureau of Reclamation except for two short periods during slack work. In October, 1936, he was appointed an inspector on concrete construction, being employed successively in taking samples in the mixing plants and later in testing samples in the concrete control laboratory.

In May, 1937, Mr. Dunlop was transferred to the Administration Office of the project to assist in the computations of triangulation and of the flood-line traverse of the Columbia River Reservoir, serving in this capacity until the work was completed in November of that year. In April, 1938, he returned to the concrete control department as inspector, and was in the employ of that department at the time of his death.

Mr. Dunlop was very studious, and was always awake to any opportunity to improve his efficiency in his work or to gain additional knowledge in his chosen branch of engineering.

He was killed on October 10, 1940, when a bullet from an accidentally discharged rifle ricocheted and struck him as he lay sleeping in an adjoining room. He is survived by three brothers and four sisters, and by many friends made from contacts in his work.

Mr. Dunlop was elected a Junior of the American Society of Civil Engineers on April 12, 1937.

JOHN PAUL HARTMAN, Jun. Am. Soc. C. E.[1]

DIED JUNE 7, 1940

John Paul Hartman was born in Toledo, Lucas County, Ohio, on February 1, 1908. He was the son of Martin and Hazel (Fenstermaker) Hartman, parents of unusual strength of character, who instilled in their son the highest principles of integrity.

After completing his studies in the public schools of Toledo, he was graduated from the Toledo Woodard Technical High School in June, 1926, and entered Purdue University, at Lafayette, Ind. He was graduated from the latter institution in June, 1930, with the degree of Bachelor of Science in Civil Engineering. He took special postgraduate work in soil mechanics at the Massachusetts Institute of Technology, in Cambridge, Mass., and upon completion of the work in December, 1936, obtained the degree of Master of Science in Civil Engineering. He was a member of Delta Tau Delta fraternity.

Mr. Hartman began the active practice of his profession in May, 1934, as a computer in the U. S. Engineer Laboratory in the Zanesville, Ohio, District.

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[1] Memoir prepared by Hobart Delos Fox, U. S. Engr. Office, Denison, Tex.

His duties pertained to soil tests, photoelastic apparatus, and model studies. From Zanesville, he was transferred to the U. S. Engineer Office at Fort Peck Dam, Mont., in 1935, as an assistant engineer, and given supervision over soil tests, photoelastic research, and model studies in connection with the Fort Peck Earth Dam. In 1938, he was transferred to the U. S. Engineer Office at Louisville, Ky., as an associate engineer in charge of the soils section. From there he was temporarily recalled to Fort Peck. In January, 1939, he was transferred to the U. S. Engineer Office at Denison, Tex., as an associate engineer (soils mechanic), and in this capacity he directed work in the Soils Laboratory and completed the investigations for the Denison Dam. In April, 1940, he received a promotion to engineer (soils mechanic) and officiated as district soils technician in charge of soils problems in connection with the embankment, highway, and railroad relocations.

Mr. Hartman perfected and secured a patent in April, 1940, on a method and means of pressure-grouting gravels, sands, silts, and mixtures thereof, through the use of a multiple-jet grouting tool, having a combined jetting and sawing action. It was designed to produce a uniform and continuous grouted volume that would be impervious to the seepage of water.

Mr. Hartman was basically an intelligent, hardworking leader, possessing force, tact, and the ability to organize and train subordinates. It is significant of the confidence shown in his shrewd ability to make quick, accurate decisions, based on clear thinking and sound judgment, that he was chosen in this connection to investigate the foundation safety of the two largest earth dams in America. He was a diligent student of engineering practice and was widely read on matters pertaining to soil mechanics.

Major Lucius D. Clay, M. Am. Soc. C. E., district engineer, when informed of his death, made the following personal contribution in token of the services he had rendered:

> "Mr. Hartman was recognized as an outstanding expert in earth fill construction, and his work has contributed materially to our present knowledge of soil mechanics. He has held a key position in this District since its formation and personally designed the earth section for the main dam. Mr. Hartman, besides being an indefatigable worker, endeared himself to his associates through his exceptionally happy and buoyant disposition. His death is a sad loss to all of us who have been associated with him in the design and construction of the Denison Dam."

In further commemoration of his services, an official memorandum was issued on June 17, 1940, designating the roadway which leads from the main highway to the Denison Dam site as Hartman Road.

Death is a medium through which one may obtain the perfect perspective that Man is Good. It was as a friend and fellow associate that the writer knew Mr. Hartman—retiring, thoughtfully considerate, and a most delightful companion. His estimation of people might be whimsical or humorous, but never unkindly. He was a man who always looked for the best in others and who was intensely loyal to all of his friends and associates. A man of moderate habits, with a consistent interest in the domestic tranquillity of his home and family, his spare time interests were centered around cabinet making, motion pictures, reading, and gardening. He was definitely not the rugged outdoor type, but he played golf and softball.

On September 1, 1934, he was married to Louise Elizabeth Eidman at Columbus, Ohio.

He died on Friday, June 7, 1940, at 12:54 a.m., in the Long-Sneed Hospital in Denison, after three weeks of illness. Surviving are his widow; one son, John Paul Hartman II; a daughter, Lee Ann Hartman; and his parents.

Mr. Hartman was elected a Junior of the American Society of Civil Engineers on December 9, 1935.

EMERY DONALD OLSON, Jun. Am. Soc. C. E.[1]

DIED AUGUST 31, 1940

Emery Donald Olson, junior engineer with the Bureau of Reclamation, United States Department of the Interior, at Denver, Colo., was killed in an automobile accident in western Nebraska, while en route to Sioux Falls, S. D., to visit his parents.

Mr. Olson was born in Lake Crystal, Minn., on December 17, 1915, the son of Leo A. and Veronica C. Olson. He was graduated in December, 1936, from the Iowa State College, at Ames, Iowa, with a degree of Bachelor of Science in Civil Engineering. From February, 1937, to June, 1939, he was research graduate assistant at Talbot Laboratory of the University of Illinois, Urbana, Ill., and was engaged in reinforced concrete slab investigations and other interesting research work. In June, 1939, Mr. Olson joined the engineering staff of the Bureau of Reclamation at Denver, and was with this organization until his death.

During his short service with the Bureau of Reclamation Mr. Olson was engaged in the design of large penstocks for hydroelectric power plants. His thorough knowledge of engineering principles and mathematics made him increasingly useful in the theoretical analysis of complex water-hammer and stress problems. He was a diligent worker, possessed a friendly disposition, and was well liked by his associates and superiors. Mr. Olson had a promising future before him and his untimely death was a great blow to his family, friends, and associates.

He was unmarried. He is survived by his parents, two sisters, and a brother. He was a member of the Gamma Alpha Scientific Fraternity.

Mr. Olson was elected a Junior of the American Society of Civil Engineers on May 17, 1937.

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[1] Memoir prepared by Peter J. Bier, Esq., Senior Engr., U. S. Bureau of Reclamation, Denver, Colo.

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OF THE

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